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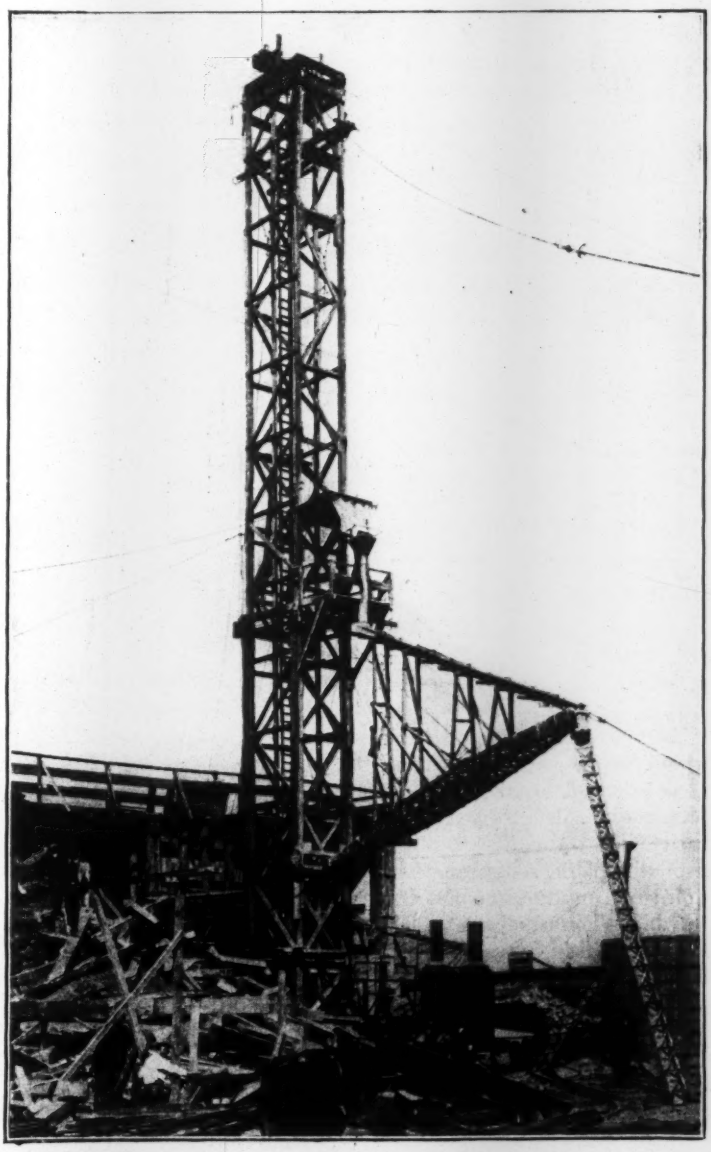
A Concrete Chuting Plant with Braced Boom and Chute

By Everett L. Jones*

To handle the 10,000 cu.yd. of concrete for the approach to the North Side Point Bridge, at Pittsburgh, Penn., with a maximum daily output of 300 cu.yd., a chuting plant of effective layout and with some special details of construction has been in operation.

The plant was located on a street which is parallel with the approach and over which all the materials had to be delivered. Four 12x12-in. mudsills 22 ft. long were laid directly on the street paving and upon these the entire foundation for the plant rested. The idea of this was to keep the mixer clear of the ground so that any water or slop from it would run down the street. Across the 12x12's, 10x10-in. timbers were next laid, and on these was spiked a 2-in. plank floor. At the front of the platform was put a 20-cu.ft. Ransome electric-driven mixer.

The charging hopper to the mixer was divided off with a sheet-iron partition so that it could be used as a measuring hopper; a second floor was built level with the top of this hopper and was used by the men charging the mixer. As this platform was some 8 ft. above the ground, an inclined 20-in. canvas belt was located so that the cement could be unloaded directly upon it from the delivery wagons and carried up and dumped onto a table at the back of the charging platform; from this table an inclined chute led into the measuring hopper so that as fast as one bag of cement was dumped another took its place. This belt was run from a shaft connected to the mixer motor through a friction drive. Only three men worked on the charging platform—one opened the cement bags, one dumped cement into the hopper, while the third measured the sand and gravel.



The Concreting Tower, Boom and Chutes Used on the North Side Point Bridge Approach, Pittsburgh

The storage bins directly above the measuring hopper had a capacity of 500 cu.yd. of sand and 100 cu.yd. of gravel, and since this material was delivered by teams a hole was dug in the street so that the wagons could be dumped through trap doors directly into the boot of a bucket conveyor, which hoisted the material into the bins. This conveyor was 45 ft. high, with an 80-ton per hour capacity, and was driven by a 15-hp. motor located on top of the bins. The water-measuring tank and the lever to charge the measuring hopper from the bins, and to charge and discharge the mixer, were located on a platform directly over the mixer. Water was let into the mixer through a quick-opening valve which would empty the measuring tank in ten seconds. Two men worked on this platform—one controlled the water and charged the measuring hopper with sand and gravel, the other charged and discharged the mixer. The concrete from the mixer was discharged into a 40-cu.ft. hoist

*Constructing Engineer, Booth & Flinn, Inc., Pittsburgh, Penn.

bucket, two batches being dumped into it before it was hoisted to the top of a 90-ft. tower, an 8x10-in. hoisting engine being used for this work. As the mixer would turn out five batches every four minutes, the engineer had to hoist only five batches every eight minutes because of the double batch.

The tower and chutes are shown in the accompanying view. The tower was constructed in the regular way with 6x6-in. corner posts halved together at the joints and well guyed with two sets of four guys each. For the distributing chutes it was desired to have a chute that could be moved quickly and without much trouble. On account of the cost and unsatisfactory results obtained from the so called free-end chutes now on the market there was built a wholly original one. The boom was built on the job from four 2x4-in. spruce sticks latticed together with 1x3-in. boards into a stiff member 50 ft. long tapering from 2 ft. 6 in. square at the middle to about 10 in. square at the ends. One end was fastened to the tower with a wrought-iron Chicago boom seat, the outer end was held up by two ¾-in. wire cables running through an equalizing sheave fastened on the tower about 18 ft. above the boom seat. On these cables were strung pieces of 2x4-in. blocks 18 in. long notched out to fit the bottom of the chute, which rested on them.

The chute was made from No. 10 iron with straight sides 10 in. deep, 9 in. wide, and a half-round bottom. The sides were flanged out on top to stiffen it, and 10-ft. sections were bolted together with splice plates to give the iron its full strength. It was additionally braced across the top with transverse angle irons and diagonal flat bars. The 50-ft. free end chute was hung from the end of the boom on a swivel joint, and to give this section additional stiffness a ⅝-in. rod was run from one end to the other through lugs riveted onto the bottom of the chute. By constructing it this way all hog rods and lateral braces were eliminated, making it much more convenient to handle, and four to six men could move the whole chute in a 100-ft. radius without any trouble. This distance could be increased by suspending fixed chutes from a cable which ran over the tower and anchored to the last arch.

The construction of this chute enabled concrete to run at a much flatter angle than the writer had ever been able to do with any chute now on the market, and to date some 8000 cu.yd. of concrete have passed through it without having it clog once. It also proved most economical for this particular case.

Booth & Flinn, Ltd., are the general contractors, while the writer is in charge of construction.

The City Tunnel of the Catskill Aqueduct

By Walter E. Spear*

SYNOPSIS—Three previous articles have (Part I, Jan. 14, 1915) outlined this work, (Part II, Jan. 21) described the shaft excavation and (Part III, Jan. 28) described the tunnel excavation. This article describes the concreting of the tunnels.

IV. Tunnel Concreting

The concrete lining of the City Tunnel was placed by the general methods worked out on the other pressure tunnels of the Catskill Aqueduct, which were described in Engineering News of June 1, 1911. Important improvements in mixing plants and forms were made in the city work, however, which led to greater speed and economy in concreting. It is intended to take up in detail only such features of the equipment and methods as differed from previous practice.

A well designed concrete-mixing plant, containing most of the good features developed by conditions on the City Tunnel, is seen in Fig. 10, which shows the plant which was built at Shaft 14 in Central Park. Gravel and sand dumped from auto trucks into the hopper at the street level were elevated by a belt conveyor into bins, from which they were drawn through chutes to gaging hoppers, thence discharged into a 1-cu.yd. concrete mixer, located in the chamber excavation close to the shaft. Cement was elevated in bags on a chain elevator or bag hoist to a storage room with a capacity of 3500 bags, located above the gravel and sand bins, from which it was dropped on signal through an 8-in. pipe into the mixer. The concrete was dumped from the mixer directly into concrete cars of 1-cu.yd. capacity on the shaft cage, lowered to the tunnel and hauled to the forms in four- to five-car trains by storage-battery tractors. This plant had a capacity of about 20 cu.yd. of concrete per hr.

On the northerly section of the tunnel, at Shafts 2 and 14, two interesting concreting plants were erected which differed essentially from others in the city.

TABLE VII. CONCRETING EQUIPMENT AT SHAFTS 2 AND 14, CITY TUNNEL

Item	Shaft 2	Shaft 14
Motor trucks for hauling aggregates	None*	Average 2 trucks of 4 to 6-cu. yd. capacity on 3 shifts
Brown locomotive crane	15-ton	None
Side dumping cars for hauling aggregate	Two 2-cu.yd. capacity	None
Storage bins, stone	300 cu.yd.	610 cu.yd.
or gravel sand or screenings	150 cu.yd.	360 cu.yd.
Belt conveyor for stone and sand	Austin, capacity, 0.4 cu.yd. per min.	Stevens-Adamson, capacity 1 cu.yd. per min.
Chain elevator for cement	None	Stevens-Adamson, 7 bags per min.
Concrete mixer	Chain belt, 1 cu.yd. capacity	Chain belt, 1 cu.yd. capacity
Concrete mixer for cement	Smith, ½ cu.yd. capacity	
Balanced cages (2) 5'x8'	Eagle Iron Works (used for workmen only)	Eagle Iron Works, for concrete
Electric hoist	Florey single-drum, 112-hp. a.c. motor	Lidgerwood single-drum, 90-hp., 220-volt d.c. motor
Electric tractors	Four 3-ton trolleys One 5-ton trolley	Six 4-ton storage batteries
Concrete cars	35 Koppel and Youngstown cars of 1½ cu.yd. capacity (long haul)	16 Koppel cars of 1½ cu.yd. capacity
Concrete forms with incline	Blaw steel forms, 1 set sidewall and arch, each form 110 ft. long 1 set sidewall and arch, each form 80 ft. long	Blaw steel forms, 3 sets sidewall and arch, each form 60 ft. long
Electric hoist on form	1 25-hp. Lidgerwood d.c. motor	One 22-hp. Lidgerwood d.c. motor

* Motor trucks hauled cement to this shaft under contract, also small amount of sand.

*Department Engineer, City Aqueduct Department, Board of Water Supply of the City of New York, 250 W. 54th St., New York City.

At each of these plants the concrete mixer was placed in the tunnel at the foot of the shaft and supplied through a 12-in. wrought-iron pipe from storage bins at the surface, with the broken stone and screenings required for each batch. Into this pipe, which was a little over 200 ft. long, was discharged in advance of the aggregates the amount of cement for each batch, the cement being first turned over with a little water in a small mixer located in the cement shed beside the shaft. The storage bins were supplied from large piles of stone and screenings crushed there from tunnel spoil. These materials were handled with a Brown locomotive crane and side-dumping cars which delivered them to a belt conveyor supplying the bins. Each plant was capable of a greater output than those dependent upon the shaft cages, something like 35 cu.yd. of concrete per hr., although an average output of less than 20 cu.yd. was realalized in concreting the sidewall and arch because of the limitations in the speed of transportation in the tunnel and in the placing of the material in the forms. The bottom section of the 12-in. wrought-iron pipe supplying the mixer at Shaft 2 wore out, and had to be replaced, but material for over 50,000 cu.yd. of concrete was discharged through the pipe at Shaft 4 without wearing more than a few holes which could readily be patched up. The principal items of the equipment of the concreting plants at Shafts 2 and 14, noted above, are shown in Table VII.

Among the important improvements made in the concreting plants on the City Tunnel may be mentioned the use of longer arch and sidewall forms, a matter which is discussed below, the raising of the sidewall forms above the springing line to decrease the amount of arch concrete, and the adoption of trucks running on the tunnel track in place of the old trucks with beveled wheels bearing directly on the concrete of the invert.

INVERT—The successive operations of lining the tunnel are shown in Fig. 11. The first step was concreting the invert, that portion of the lining on the bottom, varying from a width of 4 ft. in the tunnels of 11 ft. in diameter to a width of 5½ to 6½ ft. in the tunnel of 15 ft. diameter. Radial joints were formed for a depth of 15 in. below the finished surface in all tunnels. Beyond this depth the bottom of the tunnel outside the invert forms was leveled up to receive the sidewalls. The bonding groove formerly made in this radial joint was omitted because it was found to be difficult to properly fill this groove in concreting the sidewalls and, unless well filled, the groove decreased the water-tightness and strength of the lining. The placing of the invert was a continuous operation, the concrete being dumped from the cars into the forms from the elevated track or "high line" above the invert, the concrete spaded roughly to the shape of the invert as it began to set and finally screeded from the radial forms and given a trowel finish. The maximum slope to which the concrete was thus worked was 1 on 2½. The radius of the invert was made from 3 to 18 in. less than the remainder of the section in order to make the sidewall form bear on the edge of the invert. The speed with which the concrete in the invert was placed was limited by the amount of tunnel bottom that could be cleaned and the length of forms that could be set in advance of the concreting. Progress for one concreting plant generally averaged from 600 ft. per week, when concreting on one shift daily, to 1500 ft. per week when the placing of concrete was carried on during three shifts daily. The maximum progress was greater. In the 15-ft. tunnel, 1957 ft. of invert, 5½ ft. wide, were placed in one week of six working days from the plant at Shaft 4, working three shifts daily, and in the 11-ft. tunnel, 2281 ft., 4 ft. wide, were concreted in the same length of time with a similar organization at Shaft 21. The first record involved the placing of about 1290 cu.yd. of concrete, including the excess beyond the payment lines; the second, about 1280 cu.yd. A shift on the invert concreting was made up of 50 to 75 men, including the men cleaning up ahead of the forms.

SIDEWALL AND ARCH—The method of "trailing" arch and sidewall forms was universally adopted in placing the upper portions of the City Tunnel lining. In this method, a sidewall form and an arch form of equal length were set up together at a distance apart of 1 to 5 ft. (and on Contract 63, 20 ft.) with common incline and track, as indicated in Fig. 11, and both filled by the same organization. The cars containing the concrete, in trains of four or five cars, were delivered to the foot of the incline, hauled up by an electric hoist on the far end of the set of forms, then dumped on the platform and shoveled into the forms. This plan permitted the mixing plant and transportation system to be run more uniformly than if the arch form were filled separately, since the concrete could be diverted to the sidewalls when concreting of the arch form slowed down as the keying up of the arch began.

It was considered better practice, on the whole, to concrete toward the mixing plants, since this plan brought the sidewall form nearer the concreting plant and the delivery of concrete to the sidewall did not interfere with keying the arch, and, what is perhaps more important, when the forms were not designed to haul through, the muck from trimming and cleaning in advance of the forms could be more readily removed from the tunnel through the concreting shaft. The plan of concreting away from the mixing plant was worked out successfully, however, at several sections and was considered most desirable in tunnels through bad ground, since the workmen were less exposed to falling rock.

The concrete of the City Tunnel lining was mixed in the proportions of 1 volume of portland cement to 1½ of sand or screenings to 3 of gravel or broken stone, this richness being considered necessary to secure a maximum of water-tightness throughout. Except for the somewhat dry concrete in the arch key, the concrete was mixed with about 20 to 25 gal. of water per cubic yard, giving a wet, plastic mixture which readily filled the forms and made a dense, impermeable concrete. The concrete on either side of the key, which was banked up behind the radial boards, was mixed with less water than above and that of the key, which was still drier, had only about 16 gal. of water per cubic yard.

The shorter arch forms, 60 to 80 ft. long, were keyed from one end, the longer forms from 2 or 3 points and the closures made with the boxes shown in Fig. 12. Forms were moved within 16 to 24 hr. after they were filled.

Not the least of the preparations for concreting was the placing of drip pans in wet ground to protect the fresh concrete. These pans varied with the conditions from 4 or 5 sq.ft. to 150 or 200 sq.ft. in size. The large size was avoided if possible, since high grouting pressures applied to such areas might rupture the lining. These pans were made up of plates of sheet iron

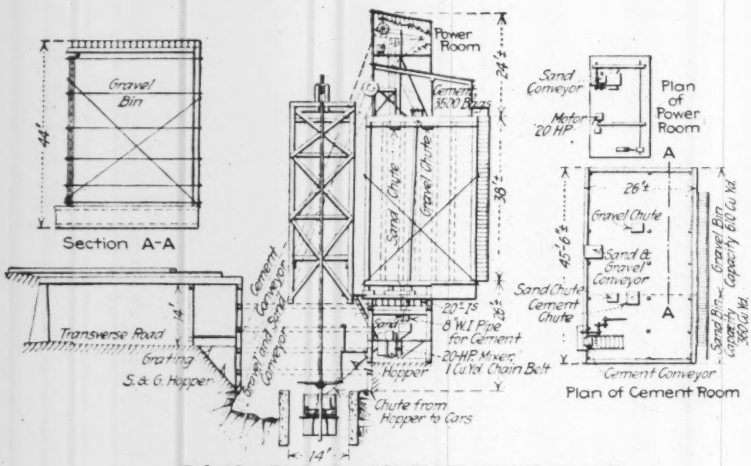


FIG. 10 - TYPICAL CONCRETE MIXING PLANT

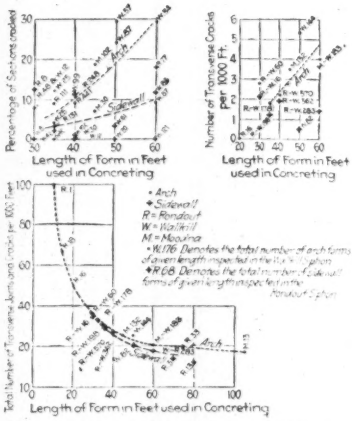
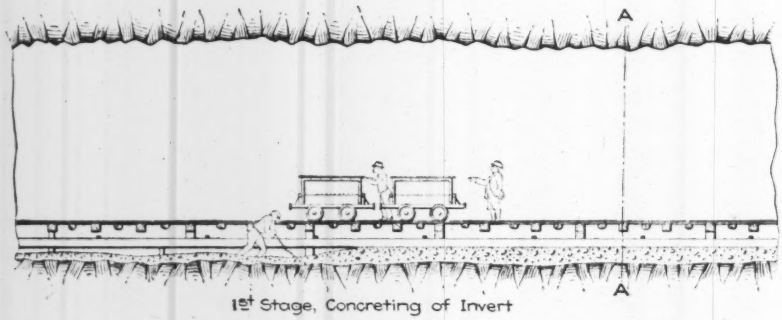
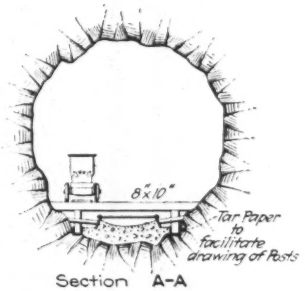
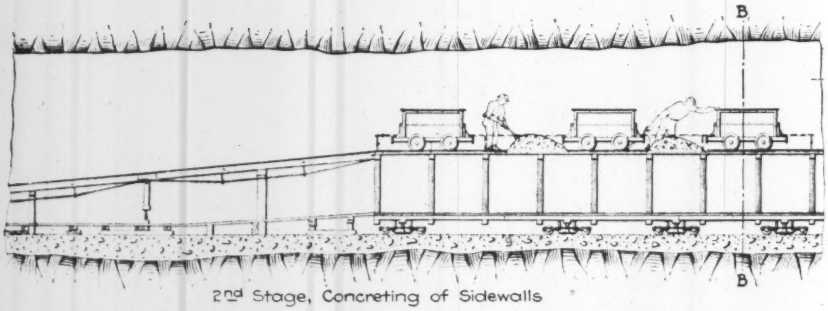
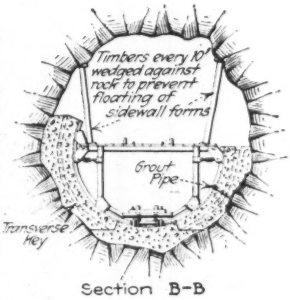

Percentage of Sections cracked
Length of Form in Feet used in Concreting
Number of Transverse Cracks per 1000 Ft.
Length of Form in Feet used in Concreting
Total Number of Transverse Joints and cracks per 100 Feet
Length of Form in Feet used in Concreting
Arch
Sidewall
R = Rondout
W = Walkill
M = Moodna
Arch
Sidewall

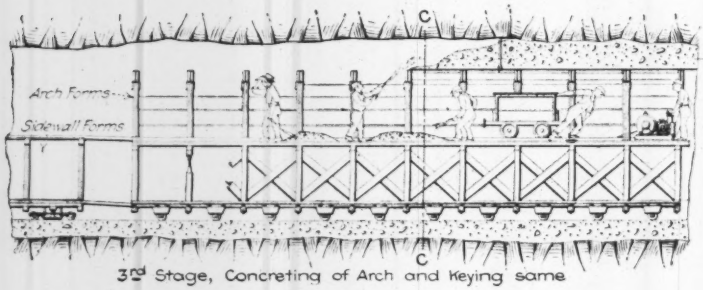
FIG 13 - RELATION BETWEEN CRACKS IN CONCRETE PRESSURE-TUNNEL LINING AND LENGTHS OF FORMS











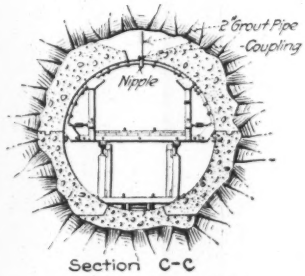


FIG. 11 - SUCCESSIVE CONCRETING OPERATIONS

of 22 to 28 gage, 24 to 30 in. wide, cut to required length, and were attached to the walls of the tunnel by nails in wood plugs driven into small holes drilled into the rock for the purpose. One or two 2-in. pipes were ordinarily placed in each pan, and carried through the forms for the drainage of the pan during concreting and for the subsequent operation of grouting, and all joints made tight with oakum to which sometimes plaster of paris was added. Pipes were also set and calked in the water-bearing seams and the water carried through the forms. In addition to the pipes in drip pans and seams, 2-in. pipes were also placed in the arch at intervals of about 25 ft. for the purpose of grouting the voids over the concrete, and others set to fill dry packing over the arch and behind the steel roof support. In placing grout pipes, the shortest possible connections were made and bends were carefully avoided as indicated in the typical sections of lined tunnel in Fig. 11.

Bonding grooves in the joint at the top of the sidewalls were, in some cases, omitted without any apparent increase in leakage. It was considered good practice to put them in, however, when the thickness of the lining at the joint was not too small, as these grooves were readily filled when concreting the arch and might possibly increase the water-tightness of the tunnel if, under pressure, there should develop a tendency to separation on this joint. By exercising care in keeping the steel forms in shape and the plates clean, excellent work was obtained in the concrete of the City Tunnel lining. An occasional shoulder at the transverse and longitudinal joints was beveled off and any bad work cut out and replaced. No plastering was permitted, but, once in a while, the natural instincts of a mason employed as a finisher led him to plaster up a crack at the joints. The writer believes that such plastering was worse than useless. Experience elsewhere indicated that a thin layer of mortar applied to old concrete would spall off after the first draining of the tunnel, if there was the slightest pressure behind it.

The relative progress made in lining the City Tunnel, the lengths of forms and the force employed are shown in Table VIII. With the forms 100 ft. or more in length, in the 14-ft. and 15-ft. tunnels of Contracts 63 and 65, only two sets of sidewall and arch forms were used at each concreting plant, one on either side, and the concreting was carried on continuously with three shifts of 45 to 65 men, one form being moved and set up while the other was being concreted. A similar schedule was maintained at each concreting plant of Contract 67, where four to five sets of forms, 60 to 80 ft. long, in the 11-ft. and 12-ft. tunnels, were concreted from each plant and three shifts of 60 to 80 men employed. In the 12-, 13- and 14-ft. tunnels on Contract 66, quite another organization was maintained as only three forms 60 to 80 ft. long were concreted from each plant and the work was ordinarily completed in two shifts. About 50 to 80 men were employed on each of these shifts and a third shift of 30 to 40 men used to make preparation for the next day's work.

Some remarkable records in concreting were made on the City Tunnel in placing sidewall and arch concrete. In the 15-ft. tunnel, 670 ft. of completed lining above the invert, comprising 2380 cu.yd. of concrete, were placed

TABLE VIII. TUNNEL-LINING DATA, PROGRESS OF CONCRETING ARCH AND SIDEWALLS

Contract	Concreting Plant at Shaft	Diameter of Tunnel, Ft.	Normal Amount of Concrete in Lining per Foot of Tunnel Within Payment Lines of Standard Type A Sections, Including Space Allowed for Dry Packing, Cu.Yd.		No. of Sets of Sidewall and Arch Forms at This Plant and Length of Each Form, Ft.	Av. Time of Filling Each Set of Forms, Hr.	Total Amount of Concrete in Sidewall and Arch Placed from This Plant		Total Elapsed Time in Months from Beginning to End of Concreting	Average Amount of Lining Placed per Month from This Plant*		Av. Cu.Yd. of Concrete Placed per Man-day†
			Full Section 10-In. "A" Line	Arch and Sidewall, Exclusive of Invert			Linear Feet of Tunnel	Actual Amount of Concrete, Cu.Yd.		Linear Feet of Tunnel	Actual Amount of Masonry, Cu.Yd.	
63	2	15	3.94	3.44	1–110	30½	5,350	19,225	6	892	3,204	1.59
					1– 80	22½	3,600	12,950		600	2,158	
	4	15	3.94	3.44	1–105	29½	5,210	20,130	6½	802	3,097	1.63
					1–150	42½	6,923	26,736		1,066	4,112	
65	6	15	3.94	3.21	2–100	36	7,130	29,310	7½	983	4,043	0.99
	8	15	3.94	3.21	2–100	36	6,544	29,023	6½	1,047	4,645	1.02
	10	15	3.94	3.23	1–100	26½	3,520	12,197	8	440	1,525	1.08
		14	3.73	3.05	1–100	26½	4,330	14,667		541	1,833	
	12	14	3.73	3.05	2–100	37	5,346	17,584	7	764	2,512	1.22
66	14	14	3.73	3.09	3– 60	14	10,165	39,165	9	1,129	4,352	1.16
	17	14	3.73	3.11	2– 60	18½	5,201	41,800	10½	1,197	3,980	1.11
		13	3.51	2.89	1– 60 to 70	18½ to 20	4,468					
		12	3.21	2.61	1– 80	24½	2,906					
					1– 60	16	1,089					
67	19	12	3.21	2.83	2– 70	17	2,427	21,362	4½	1,448	4,933	0.90
					2– 80	17	2,752					
	21	12	3.21	2.83	1– 60	19	1,266	15,367	3½	1,418	4,617	0.82
		11	2.99	2.61	3– 60	16	3,456					
	22	11	2.99	2.62	4– 60	16	6,279	18,397	4½	1,322	3,872	0.70
	23	11	2.99	2.62	2– 60	21	3,584	10,000	4	896	2,500	0.61
	24	11	2.99	2.62	2– 60	21						

Width of invert at Shafts 2 and 4, 5.5 ft.; at Shafts 6, 8 and 10 North, 6.5 ft.; at Shaft 10 South and 12, 6.0 ft.; at Shafts 14 and 17, 5 ft., and at Shafts 19, 21, 22, 23 and 24, 4 ft. Top of sidewalls was at springing line on Contracts 65 and 66; 1.5 ft. above on Contract 63 and 2.9 ft. above with radial joint on Contract 67.

* Rate of progress at Shafts 2 and 4 increased over these figures as forms approached concreting plants at these shafts.

† Based on total force employed at given plant from engineers' force accounts.

TABLE IX. SUMMARY OF GROUTING DATA, CITY TUNNEL

Contract	Diameter of Finished Tunnel, Ft.	Length of Tunnel in this Section, Ft.			Volume of Grout Placed, Cu.Yd. per Lin.Ft.		Average of Section	Approximate Av. Groundwater Head on Section, Ft.	Leakage into Tunnel, Gal. per Min.	
		Unsupported	Permanent Roof Support with Dry Packing	Total	Unsupported	Permanent Roof Support with Dry Packing			Before Grouting	After Grouting
63	15	19,214	2053	21,267	0.21	1.76	0.36	220	490	60
65	15	16,293	1366	17,659	0.35	2.40	0.51	370	218	27
	14	10,146	514	10,660	0.37	2.78	0.49	320	25	4
66	14	15,229	351	15,580	0.25	1.16	0.26	220	113	15
	13	4,561	none	4,561	0.24	No permanent support	0.24	200	0	0
	12	2,999	none	2,999	0.25	No permanent support	0.25	190	11	2
67	12	7,581	none	7,581	0.19	No permanent support	0.19	590	46	10
	11	13,507	89*	13,596	0.17	1.39	0.18	500	510	15
Total		89,530	4,373	93,903					1413	133
Average					0.26	2.03	0.34	330		

* System of light ribs and plates to protect concrete from water.

in six days from Shaft 2, and 559 ft. of sidewall and 658 ft. of arch, comprising 2287 cu.yd. of concrete in six days, were concreted from Shaft 8. In the 11-ft. tunnel, 839 ft. of arch and sidewall, comprising 2207 cu.yd., were concreted from Shaft 22 in 7 days.

The best record from one concreting plant for a month was that of 2934 ft. of sidewall and arch, or 9470 cu.yd. of concrete in the 12-ft. tunnel at Shaft 19.

Some idea of the speed with which concrete was placed in the 11- and 12-ft. tunnels may be obtained from the record made from three plants during a calendar month on Contract 67, where 23,835 cu.yd. of concrete were placed in 7503 ft. of sidewall and arch forms, where every yard had to be shoveled. A few years ago, the placing of this amount of concrete during a month in the open was considered a creditable performance for one organization. From Table VIII, it may be seen that the greatest amount of concrete in proportion to the number of men employed was put into the long forms in the 15-ft. tunnel on Contract 63. Little appeared to be gained, however, by using forms of much greater length than 100 ft. Beyond this length it became increasingly difficult to deliver the concrete rapidly enough to insure dense monolithic work.

In considering the permissible length of arch and sidewall forms for the City Tunnel, it was evident as a matter of speed and economy that a long form was better than a short one. If the concrete could be delivered to the form as fast as it could be satisfactorily placed, any increase in the length of the form would give room for more men to work and would therefore increase the amount of concrete that could be placed from a given plant, and correspondingly diminish the overhead charges. On the other hand, just how far is it possible to go in lengthening the sidewalls and arch forms, and at the same time make sure of getting the best work?

Previous to the construction of the Catskill Aqueduct, the concrete lining of large tunnels in this vicinity had not been placed in lengths greater than 40 or 50 ft., and these and even shorter lengths were adopted in lining the Rondout Siphon, the first pressure tunnel constructed. It was found in this and other pressure tunnels subsequently lined that the contraction of the concrete lining in contact with an irregular rock surface did not as a rule cause much opening up of the transverse joints. Instead of the sections drawing apart at these joints where the shrinkage of the concrete had been provided for, through bonding grooves or steel plates, irregular transverse shrinkage cracks developed between the transverse joints, even in the 30- and 40-ft. lengths, and were only absent in the shortest of the sections, a few lengths of 20 ft. This cracking of the lining appeared to be largely the result of shrinkage in cooling from the temperatures of 100° to 160° F., to which the concrete frequently rose in the forms during setting, and the temperatures of the rock behind the lining of about 60° F., and of the air to which the face of the concrete was exposed, which in the summer was not much above 60° F., and which in the winter sometimes fell below 32° F. near a down-cast shaft.

Some experiments recently made in the city indicate that this shrinkage is complete within ten days after placing, when the concrete has cooled to the temperature of the tunnel. These intermediate cracks in the lining were exceedingly fine, however, and did not appear, in general, to go through the lining, except in the upper portions of the arch which was not bonded to the rock. There was, as a rule, little trouble in making these cracks tight by grouting if the grout could be delivered over the arch to fill the space behind the cracks. Slight leakage sometimes took place through the cracks below the arch, not because the rock was necessarily exposed, but because, it seems, the resistance of the lining had been diminished through the cracking of the outer portion of the concrete. The transverse joints, even with the bonding grooves, apparently offered, on the other hand, a continuous channel from the rock to the face of the lining, and one of the greatest difficulties in grouting was to prevent the escape of grout through these joints. There was often some difficulty in placing the concrete about the steel plates which were occasionally substituted for the bonding grooves, and such joints were not always water-tight, although they perhaps held the grout better than the grooves. From these considerations and the inspection of the finished tunnels of the Catskill Aqueduct, the leakage appeared on the average to be as great from the transverse joints even after grouting and calking as from the intermediate cracks, and the avoidance of both joints and cracks seemed equally desirable.

From available data on the Rondout, Wallkill and Moodna siphons, a study was made by the writer to find to what extent the form could be lengthened without so increasing the number of intermediate cracks as to offset the advantage of fewer transverse joints. These data were plotted in Fig. 13. On the assumption that a transverse joint leaked as much as an intermediate crack, the available data in Fig. 13 showed that the advantage was with a form 60 ft. or more

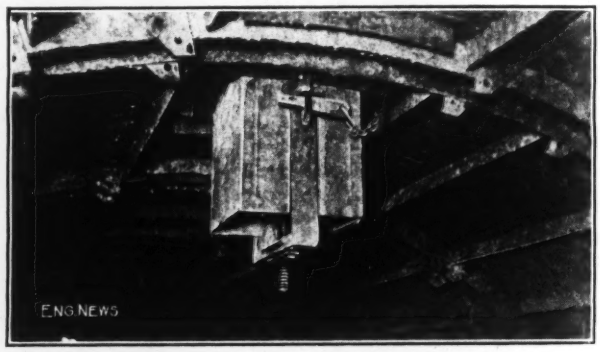


FIG. 12. CLOSURE BOX FOR PLACING LAST SECTION OF KEY CONCRETE IN CITY TUNNEL
(This box is used where key is started from more than one point in any form)

in length. The longer forms had more intermediate cracks per unit of length than the shorter ones, but, on the basis of the total number of cracks and joints together, the 100-ft. form offered a slight advantage over the 60-ft. forms. There seemed to be no objection, therefore, to the use of forms 100 ft. long, or of even greater length, if it could be demonstrated that these lengths could be filled at such a rate as to insure monolithic construction.

Forms 100 ft. in length were accordingly adopted on the City Tunnel with satisfactory results and one set 150 ft. long was tried, as shown in Table VIII. Observations on the cracking of the concrete tunnel lining placed in the city showed more intermediate cracks in some sections than were found in the Rondout, Wallkill and Moodna siphons but the comparison of the several

lengths of forms used in the city is obscured by the effects of seasonal differences in temperature. There were more cracks, for example, in the lining placed in the City Tunnel during the warmer months of 1913, even in the shorter forms, than in the concrete mixed with the same brand of cement that was put in during the following winter when the aggregates, as well as the air in the tunnel, were cool, and the concrete did not reach as high an initial temperature as in the summer. The greater amount of cracking in the city is to be explained to some extent by the richer concrete used here, since the three tunnels noted above were lined with a 1:2:4 concrete. The larger proportion of cement in the 1:1½:3 concrete of the City Tunnel lining no doubt resulted in higher setting temperatures. The records show more cracking in the linings in which the quicker-setting cements were used, since these cements doubtless caused higher temperatures of setting. On the other hand, there may possibly be some ground for the idea that porous concrete, resulting from water being forced through the lining before it sets up, has more frequently developed with the slower-setting cements.

The cost of lining the City Tunnel, including concrete materials, forms, drainage, grouting, etc., as estimated from the bid prices is seen below:

Finished Diameter of Tunnel, Ft.	Approximate Average Cost of Lining, per Foot of Tunnel
15	$52
14	52
13	48
12	51
11	51

GROUTING OF TUNNEL—The grouting of the tunnel lining followed the concreting within a period of two to three months, when the concrete had attained sufficient strength to resist high grouting pressures. Air-stirring grouting machines of the Canniff type, holding about 25 gal., were generally employed for this work, though a few mechanically stirred Cockburn machines of the same capacity were tried. For low-pressure work, by which the voids about the lining were filled, air direct from the compressor plants was used on Contracts 66 and 67; for the high-pressure work, the air pressure was raised by means of auxiliary high-pressure air compressors or "boosters." Small motor-driven compressors for both the low- and high-pressure grouting were tried out on the other two sections of the tunnel to avoid the expense of operating the large compressor plants on the surface and the cost of piping air through the tunnel. Some of these machines, of the Ingersoll-Rand type, delivered about 130 cu.ft. of free air at pressures of 200 to 350 lb. per sq.in.

For filling the voids in the dry packing and the cavities and shrinkage spaces left over the arch concrete, the grout was mixed in the proportion of 1 of cement to 1 of sand, with an equal volume of water, and forced in under pressure of 80 to 100 lb. or more per sq.in., depending on the ground-water head. Neat cement was, however, employed in filling the drip pans and other thin cavities. No masonry cutoff walls were built to stop the grout, except where dry packing was to be filled, and no attempt was then made to make them tight at the crown of the arch. Work was started at some favorable point where the grout would of itself make a cutoff and carried steadily on, connecting to each pipe in turn. On completion of the low-pressure grouting, neat-cement grout, generally mixed in the proportions of 4 to 8 volumes of water to one of cement, though sometimes containing as much as 15 volumes of water to one of cement, was forced into many of the pipes previously grouted and into the deep-seated pipes under pressures of 250 to 300 lb. per sq.in. to fill the small spaces and seams in the rock about the lining. In Table IX is shown the amount of grout placed in the several sections of tunnel, with the effect of the grouting on the inward leakage. It may be seen that in a normal section of tunnel with no dry packing to be filled, from 0.2 to 0.4 cu.yd. of grout per lin.ft. of tunnel was required. Of this not more than 10 to 15 per cent. was placed under high pressure. Where wide breakage occurred, and much dry packing was to be filled, from 1 to 2.5 cu.yd. of grout per ft. of tunnel were put in. The cost of grouting the tunnels ran from $2.50 to $3 per ft., including the payments under the several items of the contracts for plant, materials and labor.

COST OF THE TUNNEL

Until the City Tunnel is finished, the cost of construction can only be approximately estimated, but as a rough figure the tunnel will cost, exclusive of administration, in the neighborhood of $20,000,000, as follows:

Preliminary investigations		
Borings, Contracts 38, 73, 98	$171,942	
Surveys and studies	189,357	$361,299
Real estate		
Awards for tunnel easement and shaft property	314,414	
Expenses of condemnation	35,904	350,318
Construction (estimated approximately)		
Tunnel Contracts 63, 65, 66, 67	18,130,000	
Equipment contracts	748,000	
Superstructures (not yet awarded)	110,000	
Engineering	900,000	19,888,000
Total		$20,599,617

The probable units costs of the several sizes of tunnel in the city on the basis of the estimated final quantities and the contract prices are shown in Table X.

TABLE X. ESTIMATED COST OF THE CITY TUNNEL

Diam. of Tunnel, Ft.	Length of Section of This Diam., Ft.	Number of Shafts For Construction Only	Number of Shafts For Waterway Access and Drainage	Depth of Shafts, Ft. From	To	Total	Cost of Tunnel per Ft., Based on Probable Final Quantities and Unit Prices Paid Exclusive of Shafts and Equipment	Total, Including All Equipment
15	38,926	1	8½	218	477	2910	$142	$188
14	26,240	0	7	220	447	1954	148	197
13	4,561	0	1	204	224	214x	145	181
12	9,080	0	2	204	749	1005x	140	203
11	15,096	0	4½	318	757	2305x	137	246

x From original profile on which payment is made.

PERSONNEL

The City Tunnel is being constructed by contracts made by the city through the Board of Water Supply, of which Charles Strauss, Charles N. Chadwick and John F. Galvin are Commissioners, J. Waldo Smith, Chief Engineer and Alfred D. Flinn, Deputy Chief Engineer. The designs were prepared by Thomas H. Wiggin, Senior Designing Engineer in Headquarters Department, of which Thaddeus Merriman is Department Engineer. The supervision of the work in the field is directly in charge of James F. Sanborn and William B. Hunter, Division Engineers, respectively, of the Manhattan and Bronx Divisions of the City Aqueduct Department, under the general direction of the writer and Jacob S. Langthorn, Division Engineer, at the head of the Executive Division. The contractors on the four sections previously referred to are: Contract 63, Mason & Hanger Co.; Contract 65, Pittsburg Contracting Co.; Contract 66, Grant Smith & Co. and Locher; Contract 67, Holbrook, Cabot & Rollins Corporation, Geo. B. Fry and Thomas B. Bryson.

Building a Permanent Railway Track on a Pile Foundation

The plant of the Milwaukee Coke & Gas Light Co., of Milwaukee, Wis., includes a radial conveyor bridge which swings through an arc of about 185° and has its outer end supported by a tower traveling on a track of about 360-ft. radius and 1200 ft. in length. The conveyor bridge has a main span of 360 ft. and a cantilever extension of 120 ft. The track has four rails, and the tower is mounted on two 16-wheel trucks about 60 ft. c. to c. Each truck carries a load of 250 tons and has a wheelbase of 14 ft. 6 in.

The track was built in 1905 on a pile and timber trestle, with bents spaced about 6 ft. 9 in. The piles were 50 to 60 ft. long. On account of the expense of repairs and renewals, a more permanent structure was desired. The design adopted provided for cutting off the old piles at about the ground-water level, driving one additional line of piles, and then building upon this foundation a trough-shaped deck of reinforced concrete. On this deck is a fill of slag which carries the railway track. This track has 12 in. of stone ballast under the 10-ft. oak ties. A cross-section of the structure is shown in Fig. 2.

The work was done in two sections, each consisting of half the length of the structure, so that the conveyor bridge could be kept in service. For the concreting, the mixer was mounted on the middle of a flat-car, the cement (in sacks) being placed at the ends of the car. Sand and crushed slag were carried in two gondola cars, one at each end of the flat-car, and the materials were wheeled to the mixer on runways at the sides of the cars. From the mixer the concrete was spouted directly into place. This plant was moved around the work by means of a locomotive crane. The crane had also removed the old timbers, and was used later to haul cars of crushed slag for filling and to deposit this in place with a clam-shell bucket.

The regular force on the concrete mixing force outfit was as follows: 1 foreman, 1 mixer operator, 1 man handling cement, 2 men wheeling sand, 5 men shoveling slag, 2 men wheeling slag, 3 men distributing concrete, 1 locomotive crane operator, 1 fireman.

A steam hammer drove 170 footing piles (averaging 50

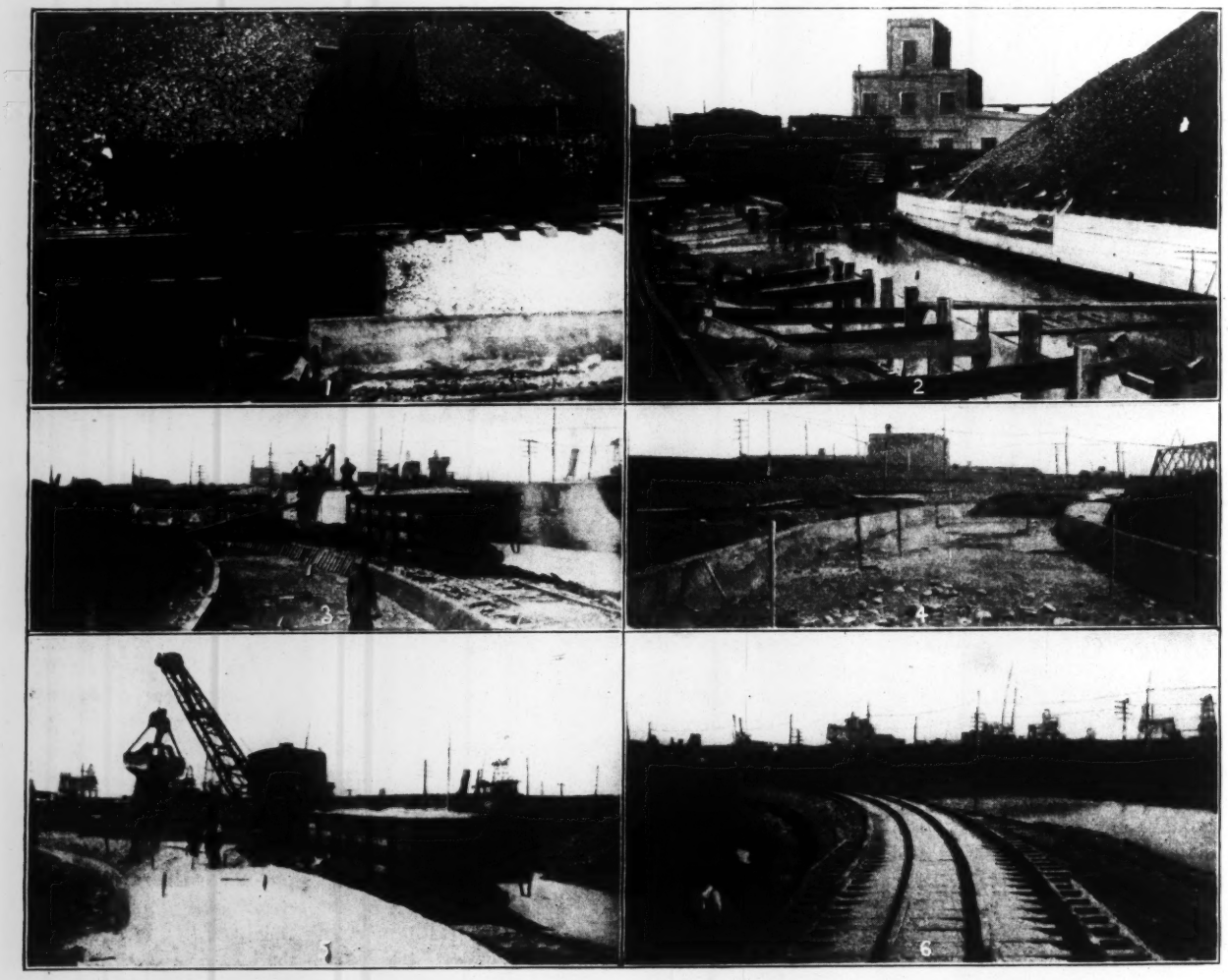


FIG. 1. CONSTRUCTION WORK ON THE CIRCULAR TRACK FOR A CONVEYOR BRIDGE; MILWAUKEE COKE & GAS LIGHT CO.

(1) One leg of bridge tower standing partly on the old trestle and partly on the new ballasted track. (2) Forms for the circular concrete deck. (3) Concreting outfit. (4) Completed trough-shaped concrete deck. (5) Placing the slag fill and stone ballast. (6) Completed four-rail circular track.

ft. long) in 15 working days. Then the track force removed the rails, and carpenters and laborers removed the trestle, the timbers being handled and loaded by the locomotive crane. The crane then excavated between the piles. The 430 pile heads were cut off by four laborers, averaging 90 piles per 10-hr. day. This was piecework, at 10c. per pile.

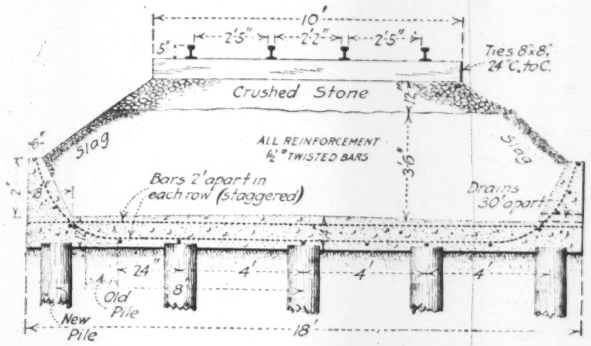


FIG. 2. CROSS-SECTION OF TRACK WITH TROUGH BASE

All the work was done by the coke company's own force (except that the new piles were driven by contract), under the direction of G. W. Mackie, Master Mechanic. This novel type of structure was designed by J. F. Jackson, Vice-President of the Wisconsin Bridge & Iron Works, Milwaukee, Wis.

The Sanitation of Iquitos, Peru*

BY G. M. CONVERSE†

A sanitary campaign for the improvement of the public health of Iquitos was begun in January, 1913, and continued until July 1, 1914.

The death records for the 10 years preceding 1913 show an average annual mortality rate of 40.56 per 1000 inhabitants, increasing to an average of 46.58 for the last three years and to 49.52 for 1912.

During the first year of the work the death rate fell to 28.88, and during the first six months of 1914 to 21 per 1000 inhabitants.

Iquitos is a Peruvian port for ocean-going vessels, 2300 miles from the mouth of the Amazon River. It lies less than 4° south of the Equator, on a bank composed of clay and sand, facing a branch of the river about half a mile wide and separated from the main stream by a long and narrow island; it is otherwise hemmed in by swamps and impenetrable jungle.

The average temperature the year round is about 30° C. (86° F.), with high humidity, the rainfall during the first nine and a half months recorded being 2,349.11 mm. (93 in.) It rained on 182 days out of 305 days recorded.

The town has 12,754 inhabitants, mostly Indian. In lieu of a sewer system it has about 18 km. of open drains —a succession of mosquito-breeding pools. There is no public water-supply. Its streets are unpaved, many of them are boggy and marshy; some are covered with a rank growth 3 to 4 ft. high, others with both water and vegetation. It has no hospital and no public clinic.

The plan had been for the writer to take up the medical problems and conduct the necessary campaigns in conjunction with an engineer who was to draft plans, furnish specifications, construct a sewer system, install a public water-supply, and pave the streets. Owing to a financial crisis produced by the low market price of crude rubber (this article having been the *raison d'être* of the town) the engineer's portion of the program came to an abrupt termination with the completion and official acceptance of his plans and specifications.

In the course of this relation it may then be remembered that, although our conduct of operations was necessarily influenced by the proposed program, no change whatsoever has taken place in the condition of the sewers,

STREET IN IQUITOS

streets, or water-supply, and therefore that such change as has occurred in the public health of the town can be fairly attributed solely to the measures adopted.

YELLOW FEVER

For various reasons the first work to be done was the eradication of yellow fever.

In the absence of a public water-supply, sewerage, and even of official levels in any part of the town, it was not found possible to order work of a permanent nature done; a householder could not be compelled to do away with the roof gutters which collected his water, nor with barrels and tanks which stored it, when there was no other way for him to obtain his supply of water. Permanent work of any character was very expensive owing to the high cost of materials and labor.

The measures adopted for the control of yellow fever at Iquitos may be summed up as follows:

1. Mosquito-proofing of all artificial containers of water.

2. Making all other collections of water unfit for the development of the larval stage of Stegomyia and, incidentally, of other mosquitos, by the addition, as done at Panama, of larvacide, at intervals shorter than the time required for the completion of the pupal stage.

3. General cleaning up of houses, yards and gardens, the removing therefrom of all unscreenable containers of water, and the cutting of grass, bushes, etc., in yards and gardens, which was ordered for the sole purpose of permitting an inspector to note, without loss of time, the presence or absence of collections or collectors of water. With the exception of an infected steamer no fumigation was done.

*Condensed from "Public Health Reports," Nov. 13, 1914.

†Acting Assistant Surgeon, United States Public Health Service, in charge of the sanitation of Iquitos.

Standard Practice Instructions for Concrete Testing Laboratory

By Ralph E. Goodwin*

SYNOPSIS—Details of practice of concrete-testing laboratory of the Public Service Commission of New York City, which has charge of all concrete testing for the enormous new subway system. Tests include investigation of aggregate and of field-made concrete, but not of cement. Rules followed by one of the few laboratories that consistently test aggregate and field concrete as a routine job.

The Public Service Commission for the First District, State of New York, at present has under supervision the construction of more than fifty contracts for new subway and elevated railroads in New York City, on which the engineering and inspection are performed by its own organization. It also has general supervision of a large additional amount of rapid transit railroad construction going forward under the superintendence of the Interborough Rapid Transit Co. and the New York Municipal Railway Corporation.

All materials of construction are inspected at the mills, shops and plants where they are produced. The inspection of the vast amount of material required for this extensive work is in charge of the Division of Materials Inspection, which, coincident with the development of the work, has expanded from a small department to one now employing over 100 men. The present paper deals with the work of one section of the division, namely, the Concrete Section.

The work of this section is two-fold. It maintains inspectors at plants which ship concrete aggregates to New York for use on Public Service Commission work, and a laboratory exclusively for the testing of concrete and concrete aggregates. So far as the writer is aware, this is the first time that inspectors have been permanently stationed at sand, gravel and crushed-stone plants, and also the first time that a complete laboratory has been devoted to the testing of concrete and concrete aggregates (not including cement, which is tested elsewhere). This paper plans to describe the technical methods followed in the laboratory. A second paper will give specifications and tests for aggregates and results of tests on field concrete.

The work performed by the laboratory aside from executive and clerical duties consists of field tests of concrete and laboratory tests of fine and coarse aggregates. One

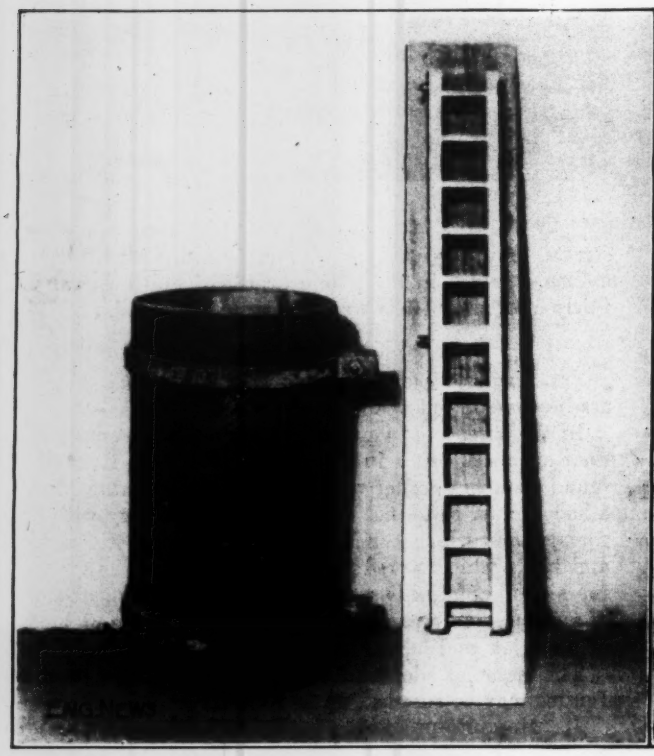


Fig. 1. Molds Used for Testing the Concrete for the New York Subway Extensions
(On left the 8x16-in. field test specimen; on right the 2-in. cube for aggregate tests)

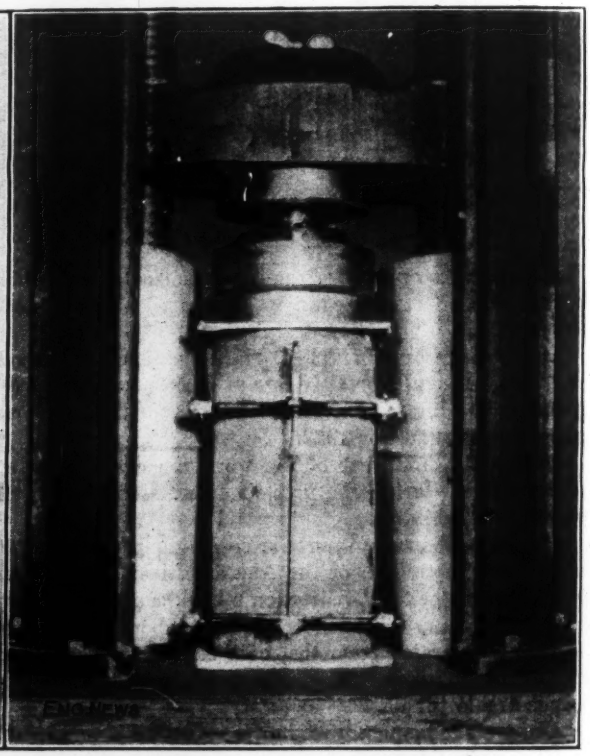


Fig. 2. 150,000-Lb. Riehle Compression Machine with 8x16-In. Test Piece

*In charge of tests of concrete and concrete aggregates, Public Service Commission, 154 Nassau St., New York City.

squad of men is constantly employed in making test specimens of concrete actually being used on the work and in taking samples of the aggregates in use for additional tests. The inside force tests specimens made by the field party and aggregates which the latter has collected, samples and tests sand from the excavations on the work at the request of Division Engineers, and performs such special tests as are necessary to assist in writing specifications and to be of general service to the work.

Equipment

The field party uses a 1-ton motor truck for transportation of men and materials between the laboratory and the work. The test specimens weigh about 70 lb. each and in addition heavy molds, large quantities of aggregates and large pieces of concrete must be transported to the laboratory from distant parts of the city.

Molds—The molds for concrete-test specimens are 8 in. in diameter by 16 in. high, as recommended by the Joint Committee on Concrete and Reinforced Concrete (Fig. 1). They are made from standard 8-in. iron pipe split longitudinally on one side. It is believed that these are in every way equal to much more expensive molds which are drilled to exact size. The variation in the diameter of the pipe is negligible.

Testing Machine—A Riehle Universal two-screw machine of 150,000 lb. capacity is used for the compression tests. This capacity is usually sufficient, though it is not rare for it to be exceeded by the strength of an 8x16-in. concrete cylinder.

Compression Blocks—A suspended block type is used on the moving head of the machine to obtain uniform compression (Fig. 2). The type of spherical-seated block which is sometimes placed beneath a compression specimen is unsatisfactory because it is top-heavy when the specimen is on it, thus making it difficult to obtain a good bearing for the top of the specimen. Additional objections to this type are that it is not sensitive and that specimens must be lifted higher to place them in the machine than when the suspended type is used.

Screens—A nest of screens for testing coarse aggregate similar in principle to a nest of sieves is used, Fig. 3. The screens are of crimped wire, having square holes with sizes of clear openings as follows: ¼ in., ½ in., ¾ in., 1 in., 1½ in. and 2 in. The apparatus is agitated by rocking. All inspectors at sand, gravel and broken-stone plants are provided with this apparatus and with scales. In addition, sieves for testing sand, balances, and graduated glass cylinders for determining silt in sand are provided for them where needed.

Scales—The scales are of the platform type, such as are used for weighing silk, with a special scale beam graduated to 0.01 lb.

Compressometer for Concrete Cylinders—(Fig. 2)—This device was designed in the laboratory because no reasonable bid could be obtained for a standard instrument. The readings are obtained from two Brown & Sharpe ratchet micrometers reading to 0.0001 in.

Concrete-Mixing Platform—A 60x$1\frac{3}{8}$-in. by 6-ft. iron plate laid on the floor forms the mixing platform.

Moist Room—A disused boiler room in the basement, with concrete floor and masonry walls, is used for a moist

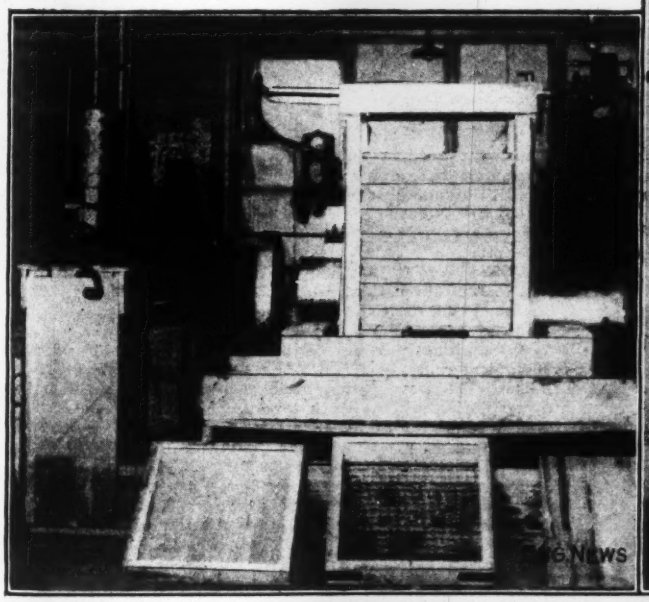


Fig. 3. Screens for Aggregate

(Sample screen for coarse aggregate in foreground; in right background nest of coarse screens in mechanical shaker; in left background fine screens in mechanical shaker)

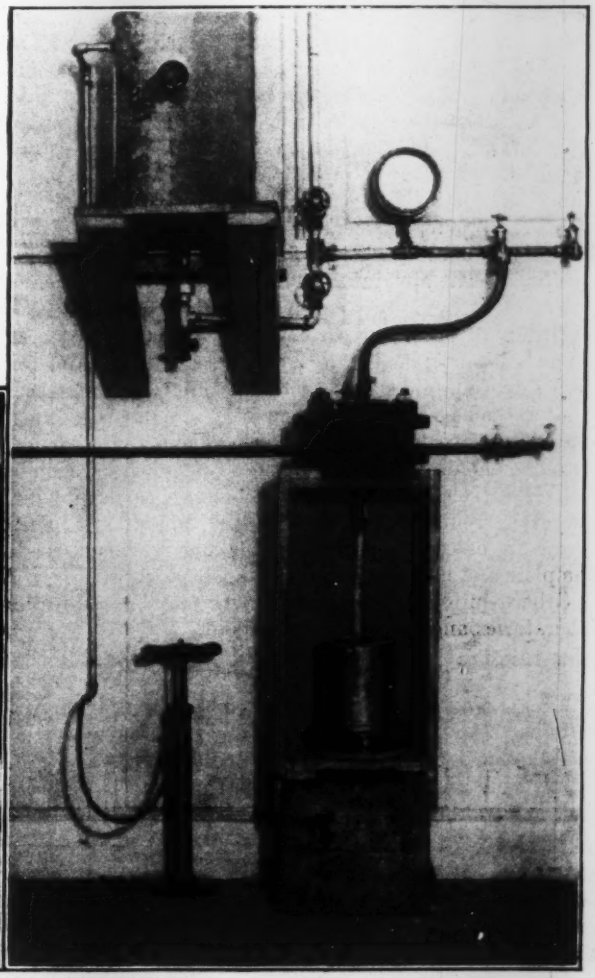


Fig. 4. Permeability Testing Machine for Concrete for New York Subway Extensions

room. Large size test specimens are stored here and sprayed with a hose daily.

TWO-INCH CUBE MOLDS—(Fig. 1)—These molds can be taken completely apart for cleaning. They are of iron and are perfectly satisfactory if cleaned and oiled after using. They are considerably cheaper than the standard pattern furnished by testing-machine companies. Wire brushes are used for cleaning them.

MECHANICAL SIEVE AGITATOR—A Howard & Morse hand-driven agitator has been adopted for 8-in. sieves (Fig. 3).

BALANCES—The balances used are: 1 analytic balance, capacity 0.001 gram to 180 grams; 2 counter scales, of which one pair is a torsion balance, capacity 0.05 gram to 1 kilogram. The torsion balance is sensitive and accurate, but the poise requires frequent adjustment.

MOIST CLOSET—A small cupboard with numerous shelves is the moist closet. The specimens are placed on the shelves in the molds and covered with wet burlap, the corners of which dip into water. The burlap is kept from contact with the specimens by stretched wires and is heavy enough to remain wet for a long time. It is thought that a more truly moist atmosphere is produced by this means than could be easily obtained in any other way.

PERMEABILITY APPARATUS—(Fig. 4)—This apparatus was made up by a plumber from standard fittings and supplies. Pressure in the tank is raised by an automobile tire pump. The specimen is cast in a section of 8-in. pipe and remains in this mold until after it is tested. The device for holding the specimen under test is composed of one 4x11-in. pipe flange and one 11-in. blank flange, the latter bored to take a ⅜-in. drip pipe. A water-tight joint between specimen and apparatus is obtained by rubber gaskets 4 in. in inside diameter by 1 in. thick. The apparatus is entirely satisfactory.

FILING SYSTEM—A 3x5-in. card file is used for keeping track of specimens to be tested. The cards are arranged chronologically, a separate one being filed for each set of specimens and for each date on which specimens are to be tested. Thus one set of specimens, of which one-half are to be broken in 28 days and the other half in three months, will have two cards in the file under the respective dates. A printed blank form is provided for every routine test made in the laboratory. These forms are of great assistance in making records accurate, complete and uniform.

Other equipment is such as may be found in all testing laboratories and requires no comment.

STANDARD PRACTICE

The standard practice of the laboratory for routine tests has been reduced to writing and is reproduced below (in small type) with some slight modifications, accompanied by such remarks (in large type) as seem to be necessary or advantageous.

State of New York
Public Service Commission
for the First District
Engineering Department—Division of Materials
Standard Practice Instructions
Fine Aggregates

Complete tests of a fine aggregate comprise:

1. Determination of per cent. retained on No. 4 square-hole sieve.
2. Mechanical analysis of portion passing No. 4 square-hole sieve.
3. Determination of silt by washing on No. 100 sieve.
4. Determination of silt by decantation.
5. Compressive tests of 2-in. cubes.
6. Microscopical examination.
7. Weight per cubic foot.
8. Voids.
9. Specific gravity.
10. Reaction to litmus.
11. Quantitative test for organic matter as indicated by loss on ignition.
12. Density in mortar.
13. Determination of insoluble silica.

It is seldom necessary to make more than the first six of the above tests and frequently only one or two of them are necessary.

When a sample is brought to the laboratory it is first given a sample number and this number, together with other data called for, is entered on the "Record of Sand Samples by Numbers." A small can is filled from the sample by the method of quartering and a sampling label pasted on the can. This sample is preserved for reference. To make the paper label stick to the can it is necessary to use tin paste applied both to the can and to the label.

1. **Per cent. Retained on No. 4 Square-Hole Sieve**—Screen all fine aggregate on No. 4 square-hole sieve before using in tests. Determine per cent. retained and throw away this portion. All material retained on No. 4 sieve is classed as coarse aggregate. The per cent. retained may be computed by weighing the whole sample as received and the residue on the sieve. The per cent. retained = weight retained × 100 divided by total weight as received. No correction need be made for moisture contained in the aggregate.

No. 4 sieve is not the same as a ¼-in. sieve. A No. 4 sieve has a 0.20-in. clear opening, while a ¼-in. sieve has a 0.25-in. clear opening. The Joint Committee recommends that a ¼-in. round-hole sieve be used. But these are more difficult to obtain than ordinary sieves and the difference in results is negligible. Either a No. 4 or a ¼-in. square-hole sieve is satisfactory for commercial work.

2. **Mechanical Analysis of Portion Passing No. 4 Sieve**—For this test the following sieves are used: Nos. 8, 16, 30, 50, 100. In all sieve tests for which the sample must be dried there is danger of a separation of fine from coarse particles after drying. For this reason always weigh out a separate portion of moist aggregate for each determination. This should weigh not over 10% more than the weight of dry aggregate desired.

Separate approximately 150 grams of the sample by method of quartering, using portion which has passed a No. 4 square-hole sieve. Weigh out 110 grams of this.

Dry beneath gas burner at 100° C. until dust rises when aggregate is shaken. Aggregate will dry quicker under the gas stove than in the drying oven.

Weigh out exactly 100 grams of this and put it in the nest of sieves. Extra care must be taken in dividing dry aggregate to prevent the separation of fine from coarse. In using the torsion balance the higher the adjustable weight is placed the greater the accuracy obtained.

The amount of sieving should be 200 revolutions of the crank. Oil the machine occasionally. Placing fine shot on the No. 50 and No. 100 sieves to assist the fine material to pass these sieves will not change the results appreciably and is unnecessary.

Record the weight retained on each sieve and the weight passing the last sieve in the last column of the report. The sum of these quantities should equal 100 grams. This will make a permanent record of the accuracy of the test.

Finally, compute the per cent. of the whole which passed each sieve and plot the graph.

The fine-mesh sieves must be used with care. Do not lay them down with the wire cloth touching anything, and do not rub them with the fingers; use a brush.

For method of plotting mechanical analysis curves see "Concrete Plain and Reinforced," Taylor and Thompson, p. 193.

3. **Silt by Washing on No. 100 Sieve**—Separate 220 grams of moist sand by quartering and dry at a temperature low enough to avoid baking any clay or similar matter, which would thus be rendered incapable of passing the sieve. If time permits, dry at room temperature by spreading on a plate of glass. Use 200 grams of dry sand for the test. Extra care must be taken in dividing dry aggregate to prevent separation of fine from coarse. After weighing allow the material to soak in water for a few minutes in order to soften hard lumps. Wash very thoroughly under a gentle stream of water (a violent stream would injure the sieve) and dry under gas

burner. Be extremely careful to avoid heating sieve to a high temperature, which would melt the solder holding the sieve cloth to the frame. When dry weigh the residue on the sieve. The per cent. of silt is the loss of weight × 100 divided by 200. State result clearly on report as follows: "Per cent. of silt by washing through No. 100 sieve = ——."

This washing test is considered necessary because if there is clay in the sand it forms a coating on the grains, which adheres and does not pass through the sieves in a dry state. Washing is necessary to obtain the true silt value.

4. **Determination of Silt by Decantation**—For field use only. Apparatus needed: Two or more 100 c.c. glass cylinders graduated to 0.5 c.c.

(1) Take a fair sample of the sand to be tested by mixing portions from different parts of the pile. Mix thoroughly and spread out into a flat heap. Take opposite quarters of the heap, mix, and spread as before. Repeat until the original sample is reduced to a small quantity.

(2) Take approximately 20 c.c. of the small sample in one of the graduated cylinders. Add approximately 30 c.c. of lukewarm water and stir with a wire for thirty seconds.

(3) Allow to settle for thirty seconds. Then decant the water into an empty cylinder. Stir up the sand left in the bottom of the first cylinder with a fresh portion of water and repeat. Do this four times, then place the two cylinders on a level surface for one hour.

(4) After one hour record the height of silt and sand which has settled to the bottom of the cylinders.

(5) Compute the per cent. of silt as follows:

SILT DETERMINATION SAND SAMPLE No. 45
Route 2, Section 20
P. Smith Contracting Co.

From subway excavation at 81st street. Sampled by J. Jones. Remarks: This sand being stored at 83rd St. for future use. Tested by J. Jones.

19.0 c.c. = volume of clean sand remaining in cylinder No. 1.
1.6 c.c. = volume of silt in cylinder No. 2.

20.6 c.c. = total volume.
8% = per cent. of silt = 1.6 × 100 ÷ 20.6.

Several duplicate tests should be made and the average value used.

The above is a method frequently used in the field. It is thought that decantation is an improvement over the old method of allowing the silt to settle on top of the sand in one test tube, as in the latter case some of the silt settles back between the grains of sand.

5. **Compressive Tests of 2-In. Cubes.**—Use methods recommended by Committee on Uniform Tests of Cement, of American Society of Civil Engineers, except where the following instructions require otherwise.

To avoid the removal of any coating on the grains, which may affect the strength, fine aggregate must not be dried before being made into mortar, but must contain natural moisture. The percentage of moisture may be determined on a separate sample for correcting weight. The weight of moisture is counted as a part of the water used for mixing and not as a part of the weight of the sand. Proportions are to be 1 : 3 by weight. Since the sand is weighed in a moist condition, the actual weight will be three times the weight of cement plus the weight of the moisture contained. The cement should be screened through a No. 30 sieve and then worked over dry with a trowel to break up any remaining lumps. Mortar is to be made of the same consistency as that of Ottawa sand mortar when mixed with 60% of water above what is required to produce "normal consistency;" e.g. if 10% of water is required to make Ottawa sand mortar of "normal consistency," then 16% of water will be required to make Ottawa sand mortar of Public Service Commission consistency. Since the amount of water necessary to produce a given consistency varies with different aggregates, the only known way to determine the correct amount of water to use is experimentation. The temperature of the gage water must be about 70° F. Mortar must be mixed hard with the hands for the time specified and thoroughly worked into the molds with hands and trowel. Place a small amount in each mold and work it well to force out air bubbles before adding more mortar. Continue in this manner until the molds are full.

The specimens must be kept in air that is actually moist for the specified time, then marked and weighed and removed to storage tanks. For marking specimens use black crayon pencil or red-lead paint. Mark each specimen with the sample number and the number of the specimen, thus, $\frac{2}{1}$. Standard sand specimens are marked with the same numbers as specimens of the sand to be tested and an "S" is added to indicate "Standard" sand specimen. The storage tanks must be emptied and refilled with fresh water every two weeks. File report on proper form and file cards to show when the specimens are to be broken.

Ten specimens are made in each test, to be broken as follows:

4 after 3 days.
3 after 7 days.
3 after 28 days.

The younger the specimen the greater the care that must be taken to break it at the time that it is due. Three-day specimens must be tested within one or two hours of the correct time.

Ottawa-sand specimens of the same consistency are to be made with each test.

In crushing cubes observe the following important instructions:

Specimens which vary in weight more than 3% from the average should be rejected.
Cubes are to be kept in water up to the time of crushing and must not be allowed to dry out.
They are to be laid on their sides for testing.
Place a double thickness of blotting paper above and below.
They must be placed exactly in the axis of the spherical block. Use a template for centering them.
As the testing head descends keep the spherical block in motion with the hands in such a way as to insure a good bearing.
The pressure should increase uniformly at such speed that it will require between one and two minutes to crush each specimen.

Since difference in consistency greatly affects the strength at early periods, it should not be considered serious if the results fall below those for Standard sand specimens by the following amounts or less:

At 3 days 10%
At 7 days 5%.
At 28 days 1%.

No tensile tests are required, because it is thought that compressive tests more nearly represent the conditions of the work and that modern practice is tending toward compressive tests.

A wet consistency is used in preference to "normal consistency." This is due to the fact that some evidence has been presented to show that a sand which is satisfactory in "normal consistency" may fail in wet consistency. (See Reinke, "Proc. Am. Soc. Testing Materials," 1913, p. 797.) Also wet consistency more nearly represents working conditions. No device for determining consistency is used, although some experiments along this line have been made. (See Chapman, "Proc. Am. Soc. Testing Materials," 1913, p. 1045.) The judgment of the operator is considered to be a sufficient guide for commercial work.

6. **Microscopical Examination**—"An examination of the grains of dirty sand with a microscope will frequently show a crust of organic matter on the grains which is not readily brushed off."—"Concrete Plain and Reinforced," Taylor and Thompson.

7. **Weight per Cubic Foot**—Use an 8x16-in. concrete cylinder mold for a measure. Place inside of this a smaller cylinder which is high enough to contain somewhat more than a measureful of fine aggregate. Fill the inner cylinder with fine aggregate and remove it, allowing the aggregate to flow out at the bottom into the measure. Strike off the top level, clean the outside of measure and scales and weigh. Weight of contents equals gross minus tare. The aggregate in the measure must now be dried. After drying, clean the small cylinder of sand sticking to the inside and use it for placing the dry sand in the measure by the same method as before. Level off the sand and determine the volume by measuring down from the top. Weigh. Preserve the aggregate for use in the determination of voids.

The above method is due to C. M. Chapman, ENGINEERING NEWS, Feb. 5, 1914. Sufficient data are obtained in this test for the calculation of weight per cubic foot as received and weight per cubic foot dried—two very different figures.

8. **Voids**—The manner in which this test is performed has the advantage over other methods of permitting a large sample to be used. The same portion of aggregate as was used in the previous test (No. 7) is used in this. Hence its apparent volume and its weight in both wet and dry condition are already known. It only remains to determine the absolute volume of solid matter. This can be computed from the weight, assuming a specific gravity 2.65, or it can be found as follows: (1) Place the aggregate in a receptacle containing water and weigh the whole. (2) Empty out the aggregate, refill the receptacle with water to exactly the same height as before and reweigh.

Let

S = Weight of dry aggregate.
(1) = Result of first weighing.
(2) = Result of second weighing.

Then (2) — [(1) — (S)] = the weight of a volume of water equal to the absolute volume of the aggregate. The absolute volume of aggregate is computed from the weight of water. Per cent. of voids in aggregate = (Apparent volume — absolute volume) × 100 ÷ (apparent volume.)

If the second method of finding absolute volume is used data are obtained from which the specific gravity of the aggregate may be computed, though the method of the next test (No. 9) is more satisfactory.

The following details are to be observed: Use contents of measure from previous determination (No. 7), a small waste can and the hook-gage. A few inches of water is placed in the can and enough added from time to time to keep the water-level above the aggregate, which must be shaken slowly into the water to avoid entraining air in the voids. When the aggregate is all in the can raise the water-level to the point of the hook-gage as shown by the distortion of the surface film over the point. This is best noted by getting the reflection of a light on the point to be observed. Mark the position of the hook-gage on the can so that it may be replaced in the same position. Empty and clean the can, replace on scales and fill with clean water to exactly same height as before, using hook-gage in same position on can as before. The above precautions are necessary in order to obtain accurate results. Weigh the whole apparatus, including hook-gage.

9. **Specific Gravity**—Specific Gravity may be obtained by means of a specific-gravity flask or of a graduated glass cylinder used in the same way. In any case the aggregate is to be shaken into the liquid in order to avoid entraining air in the voids.

10. **Reaction to Litmus**—This test may be used when necessary to demonstrate the presence of alkalis which would be injurious to the cement.

11. **Organic Matter**—The following determinations are to be made in duplicate for the purpose of checking:

Take approximately 75 grams of sample by method of quartering. This portion should be fresh from the sample can, since sand exposed in the laboratory dries rapidly. Weigh out two amounts of 25 grams each, sand with natural moisture. Put each portion in a beaker. Mark one beaker "A" and the other "B". Cover each portion of aggregate with 60 c.c. of water at 100° F. Mark one filter paper "A" and another "B", on the back side near edge in pencil. Dry these in oven at 100° C. for 15 minutes or more.

(1) Weigh separately on analytic balances and place in funnels ready for use. Stir the sand and water in beaker "A" briskly with a glass rod. Allow to settle 30 seconds. Decant part of water onto filter "A". Stir, allow to settle, and repeat until water is all filtered. Add 30 c.c. additional water at 100° F. and repeat. Allow filtrate to drain and place washed sand under gas burner to dry. Follow the same procedure with beaker "B".

Now remove the filter papers with filtrate on them, from the funnels and place in a casserole in oven at 100° C. When partly dry spread filter papers out flat on wire gauze and continue drying in oven until further drying will produce no loss of weight. The temperature must not be raised above 100° C. Fold filter paper with filtrate on it into a small wad, being careful not to break the paper.

(2) Weigh.

(3) Weight of silt = (2) — (1).

Ignite thoroughly in porcelain crucible. If black carbon forms on side of crucible heat that part to red heat until carbon burns off.

(4) Weigh the residue.

(5) Weigh the washed sand.

Compute the per cent. of loss on ignition $\frac{[(3)-(4)]\times 100}{(3)+(5)}$

This test is seldom used because more direct results are obtained by tests of 2-in. cubes. Within the experience of this laboratory, any sand which exceeds the specifications for organic matter has also appeared dirty and suspicious.

12. **Density**—Use method explained in "Concrete Plain and Reinforced," Taylor and Thompson, p. 139, with the following exception. The weight adhering to tray and tools is best determined by taking difference between weight of mortar in glass graduate and the sum of the weights of the separate ingredients. It is inaccurate to weigh the mix adhering to tray and tools because the evaporation of water from these surfaces frequently is as great as four grams or more.

This test is seldom used for the following reasons:

(1) The density can be fairly well predicted from the mechanical analysis.

(2) The density test gives no information about harmful impurities which would affect the strength.

13. **Insoluble Silica**—This is a chemical test to be made by a chemist.

Coarse Aggregates

Complete tests of a coarse aggregate include:

1. Mechanical analysis.
2. Cleanliness.
3. Weight per cubic foot.
4. Voids.
5. Specific gravity.
6. Crushing strength of stone.

It is usually necessary to make only the first two of the above tests.

Coarse aggregates are numbered and samples are preserved exactly as is done with fine aggregates.

1. **Mechanical Analysis**—Ordinarily use a 25-lb. sample reduced from a larger sample, if necessary, by quartering. Carelessness in selecting or dividing the sample makes the test worthless. If necessary to use a smaller sample use as large a one as possible, the results being easily reduced by slide-rule. Screen in rocker 100 bumps (50 cycles). Weigh residue on each screen and record in last column of regular record blank. Record also weight of sample used. This will give a permanent record of the accuracy of the test. The sum of the weights retained on the different screens, together with the weight passing the ¼-in. screen, should equal the weight of the sample used. If a large sample is used some of the screens will be found to be overloaded. The material on these must then be divided and rescreened in such a way as to give correct results. To avoid rehandling the material have each screen marked with its tare weight and weigh residue and screen together.

2. **Cleanliness**—Cleanliness is judged by inspection only.

3. **Weight per Cubic Foot**—If the sample is very wet it should be dried sufficiently to remove surplus moisture, but not enough to dry out the pores. Be careful to use a representative portion for the determination. Pour this portion slowly into the measure from a height of 2 ft. above the bottom. Strike off the top, clean the outside of the measure and the scales, and weigh. Use an 8x16-in. cylinder mold for a measure.

4. **Voids**—The same method is used for this test as for the corresponding test of fine aggregate.

5. **Specific Gravity**—Select a number of pieces which will go into a 300-c.c. graduated cylinder and the average of which represents the average of the material. Wash off all dirt and dust. Dry thoroughly under gas burner to remove all moisture from the pores. Allow to cool. Weigh. Boil in water and allow to stand in water until cool. This will cause the pores to fill with water. Dry with a towel, weigh and place in a 300-c.c. graduated cylinder. Pour in a measured amount of water from another graduated cylinder sufficient to cover the stone. Obtain the volume of the stone in this manner and calculate the specific gravity and the absorption based on dry weight.

6. **Crushing Strength of Stone**—No test of this kind has yet been attempted.

Field Tests of Concrete

Molds—Large molds (8x16-in. cylinders) should be used. A smaller size is too small for coarse aggregates. If the edges do not register accurately when the bolts are tightened try changing the position of the straps. Iron molds are oiled with heavy machine oil. For wooden molds use tallow or Albany grease. Set the molds on level iron plates, also oiled. It is not well to set them on a piece of board because it may be wrapped and permit leakage from the specimen. Place sand or mud around the bottom of the mold to prevent leakage.

Sampling—Concrete for the specimens should be taken directly from the forms at a place where it has been rammed or

spaded to its final condition. A fairer sample can be obtained after the concrete has been spaded than before. Great care must be taken to procure an average sample. The molds should be placed as near the forms as possible in order to avoid separation of coarse aggregate from the concrete during transportation. In order to overcome the effects of such separation as may be unavoidable, distribute each pailful of concrete to several specimens. A little care in finishing the top surfaces of specimens will save much trouble when they are tested. If the concrete is very wet it will shrink and expel water after standing. The ends of such specimens may be finished with dry sand and cement after standing for a few minutes.

The molds are to be filled in not less than three layers, working each layer well with a trowel to remove air bubbles. Each test consists of six specimens, three to be broken 28 days after mixing and three to be broken 90 days after mixing.

Obtain a 25-lb. sample of coarse aggregate and 3 quarts of fine aggregate to be tested in the laboratory. Care must be used to select average samples, as otherwise it is useless to take any. Record data called for on the proper form. Record the make of mixer used and be particular to give correct consistency, since the consistency largely affects the strength. The following terms describe consistency: Dry, medium, wet, very wet, extra wet.

Use "wet" to designate wet concrete which is stiff enough to hold together. Use "very wet" in cases where the coarse aggregate settles to the bottom. Use "extra wet" to designate soupy consistency.

Storage—Specimens are left on the work two days and are then brought to the moist room in the molds. In the moist room remove the molds and paint on the cylinders the contractor's initials, test number and specimen numbers. Blue lumber crayon must not be used for marking green concrete because it will bleach and become illegible.

Specimens are stored in the moist room and sprinkled daily with the hose until two days before testing. Place two cards in the file to show dates when each set of three cylinders is to be broken. Two days before breaking each set send a notice to the Section Engineer concerned in order that he may be present at the test. Specimens must be removed from the moist room to the laboratory two days before breaking in order that they may become dry. The strength of saturated specimens is considerably below that of dry ones. The record should be as follows for 28-day specimens and similar for 90-day specimens:

Length of time in forms, 2 days.
Length of time in moist room, 24 days.
Removed from moist room, 2 days before tested.

Finishing Ends—If the ends of specimens are not true planes, or if they are very rough, or if the edges are broken, they must be finished with plaster of paris, in order to insure an even bearing. Very poor ends must be finished with neat cement. A poor end may often be detected by rocking a specimen on its base or by placing a flat plate on its top and noticing if it rocks. Sometimes a space can be seen between the plate and the specimen, indicating a poor bearing which must be corrected.

To finish the end of a specimen with plaster lay a piece of paper on a plane, smooth surface and spread plaster paste on the paper. Set the specimen in this and leave it until the plaster is hard. The plaster must be so soft that the specimen will sink into it and press from beneath itself all the plaster except a thin surface. The specimen must be set in the plaster carefully and must not be moved nor tipped after it has been set down. To finish specimens whose ends are damaged replace them in the molds or use sheet-iron molds made for this purpose. If the ends are finished with plaster, in crushing use iron plates with perfectly plane surfaces above and below to protect the faces of the testing machine.

A specimen whose ends are planes and fairly smooth need not be plastered. The ends of all specimens not plastered are to be cushioned against plain board, compo board, or some similar material. This should be turned over after each test in order that the dented side may not be next to the specimen. Soft material, like sheet lead or felt, must not be used, because when it comes under pressure it spreads laterally and introduces tensile stresses which split the ends of the specimen before the true compressive strength of the concrete is attained. It is possible that a piece of sheet iron inserted between the specimen and the cushion might partially prevent these tensile stresses from reaching and acting on the test piece. Further experiment in this direction is to be desired.

The manner in which a specimen breaks indicates the character of its bearing. In properly manipulated tests the fracture either occurs in the middle, leaving the ends intact, or it originates in the middle and spreads to the ends, the fracture of the ends being a secondary result. General Gillmore used wood for cushioning the ends of specimens of building stone because he believed that working conditions were most nearly reproduced by this method. He also carried out tests which seemed to show that at pressures below 7000 or 8000 lb.per sq.in. wood and steel cushions give the same results. (Report of Chief of Engineers, U. S. A., 1875, Part 2, Appendix II.) Since concrete will never attain to more than about 4000 lb. per sq.in., we may conclude that wood cushions can be used without affecting the results.

There are two ways in which it is possible for the cushions to affect a test: (1) By spreading laterally they may split the specimen as explained above.

(2) By their elasticity they may reduce the rate of application of the load and thus lower the ultimate strength. If a load is applied until cracks begin to develop and the specimen is then left standing under this load without increasing it, failure will nevertheless ultimately occur. This is due to the fact that small cracks grow large even though the load is not increased. But a higher strength can be obtained by rapidly increasing the load up to the point of complete failure.

Of the two effects the first is by far the most important.

Technique of Testing—Specimens are to be weighed on the day they are broken, after drying two days in the laboratory, and before the ends are finished with plaster. They must stand vertical in the testing machine so that the stress will be along the axis. If the spherical block is on the upper head of the testing machine the specimen should be set on its bottom (the square end.) If the spherical block is below, the specimen will be placed with its square end up. It must be centered exactly in line with the axis of the spherical block. Failure to do this throws the load on one side of the specimen and gives an unfair test. The effect of faulty centering is easily seen in the way in which the specimen breaks and is so great that careful attention must be given to this detail. As the testing head descends on the test piece the spherical block should be kept in motion with the hands in such a way as to insure a perfect bearing. Avoid coming down on the specimen at fast speed with a shock or jar. The rate of application of the load is the same as for testing 2-in. cubes. With an 8x16-in. cylinder in the machine this speed causes the scale arm to rise about as fast as it is possible to follow it accurately with the counterweight. Place heavy paper between the specimen and the screws of the machine in order to keep the latter clean when the specimen breaks.

The report of the test will not be very valuable unless the operator is alert to note and report important details. Notes such as the following greatly increase the value of a report:

Specimen No. 2 had excess of stone at center.
Concrete so wet that a fair sample could not be obtained.
Coarse aggregate sheared in specimen No. 3.
No bond between mortar and coarse aggregate. Latter appears as if surrounded by a film of water when mixed.
Extreme cold injured specimens while stored on the work.
Gravel used contained with sand.

A daily record of out-of-door temperatures should be kept in order that the cause of variations due to extremes of temperature may be known.

Mixing Concrete in the Laboratory

Proportions—Concrete is to be proportioned by volume in order to reproduce field conditions, but in order to secure uniformity the materials are to be weighed. Determine the weight per cubic foot of the aggregates by the methods explained under "Fine Aggregates" and "Coarse Aggregates." Then weigh out the proper amounts to give the desired proportions by volume. The weight of portland cement is to be taken as 375 lb. for 3½ cu.ft., 107 lb. for 1 cu.ft. (Public Service Commission specification.)

Mixing—Spread the sand in a layer on the platform and spread the cement evenly on top. Turn the dry sand and cement together three times. By rotating the shovel quickly in the hands in such a way that the material is shaken off the edges, a very thorough mixture is obtained. Next spread the coarse aggregate in an even layer. Weigh out 6% of water in one receptacle and in another have additional water of known weight ready for use. Drench the coarse aggregate with sprinkling pot, using part of the 6% of water. Spread the mixed sand and cement evenly on top and wet down with the remainder of the 6% of water, using sprinkling pot. Allow water to soak in. Turn together three times. If the mixture is too dry add more water. The consistency to be aimed at is such that the concrete will slump if placed in an 8x16-in.

cylinder mold and the mold removed, but not wet enough to cause trouble by flowing away from the mixing platform.

Record per cent. of water used. If extreme accuracy is required, this figure must include moisture contained in the fine aggregate. If this is included the fact should be stated on the report. A neat record of all quantities should be kept in order that the results may be verified and errors located. Place cards in the file to indicate the dates when the specimens are to be tested.

Permeability Specimens

Mix the mortar as per directions accompanying the waterproofing compound which is to be tested. Make several specimens to be tested after 7 days and several more to be tested after 28 days. In no case make fewer than eight treated specimens (total number) and eight untreated specimens for comparison. Use extra clean, coarse sand (not Ottawa sand) for these specimens in order that the waterproofing compound may receive no assistance from silt in the sand. In the absence of different directions mix the mortar to the same consistency as herein specified for 2-in. cubes. Place the mortar in the 8-in. pipe sections in the same way as specified for 2-in. cubes. Specimens are to be ½ in. thick unless otherwise noted. The iron plates under specimens must be thoroughly oiled. Strike off tops of specimens with a straightedge.

Store test pieces in moist air 24 hours, then brush both surfaces with a wire brush, mark, and place in water until tested. No paint marks (for numbering test pieces) must be placed on the surfaces where they might affect the results. The specimens are not removed from the molds until after the test.

Upon removing specimens from storage tanks for testing, brush both surfaces again with a wire brush. Test under a pressure of 50 lb. per sq.in. for at least seven hours. After the pressure is on let out any air which may be caught in the hose, etc., by means of the cocks on the upper flanges (see Fig. 4). Some means must be used to prevent water from the outside from leaking into the measuring glass, since it has a tendency to follow down the outside of the outlet pipe. This may be accomplished by means of a rubber stopper in the mouth of the glass. It is assumed that if a specimen does not begin to leak in seven hours it is impermeable.

Record results at 15 min. intervals. On report show the time elapsed from application of pressure to first leakage, the average leakage for the period of actual flow, and the maximum leakage for one hour. Express results in c.c. per sq.ft. per hour.

Alfred Craven is Chief Engineer and Robert Ridgway is Engineer of Subway Construction for the Public Service Commission. George L. Lucas is General Inspector of Materials. Laboratory tests are in charge of I. J. Elkind, and, until recently, Joseph I. Shankey was in charge of field tests.

Making Over a Small Sewage-Treatment Plant at Moorestown, N. J.

An outgrown and neglected sewage-treatment plant at Moorestown, N. J., was recently enlarged and improved by converting the original septic tank into a two-story tank and the old contact beds into sprinkling filters. The plant was designed and built in 1901 with Alexander Potter, of New York City, as engineer.

In 1912, the New Jersey State Board of Health ordered that the plant be enlarged and in the summer of that year the voters authorized $15,000 of bonds for the purpose, before any plans had been made. Mr. Potter was engaged to design and build the enlargement, which he did as best he could with the funds available. The following account of the work is condensed from a paper which Mr. Potter presented before the American Society

FIG. 2. TWO-STORY SEWAGE TANK, MOORESTOWN, N. J.

of Municipal Improvements at Boston, Mass., in October, 1914.

The original plant consisted of (1) a grit chamber 10x10 ft. square by 10 ft. deep; (2) an uncovered septic tank, of brick, 25½ ft. wide, 75 ft. long, with an average depth of 7 ft., divided into four compartments, arranged so that two, three or four of them could be used as desired, and one serve as a dosing chamber; (3) four contact beds, each 85x42 ft. in area. The plant was designed to treat from 200,000 to 250,000 gal. per day, based on an eight-hour detention period in the tank and an average rate of 600,000 gal. per acre per day in the contact beds.

In March, 1911, the State Board of Health found that the plant was receiving 500,000 gal. of sewage per day, and that there were various evidences of neglect and overwork.

CONVERTING THE SEPTIC TANK INTO A TWO-STORY TANK

To convert the old septic tank into a two-story tank (Fig. 1) the old bottom was removed, the earth excavated to an additional depth of 2⅓ ft., and the old sidewalls raised 6 in., thus increasing the depth of the tank from 7 ft. to 9½ ft. A top view of the plant is shown by

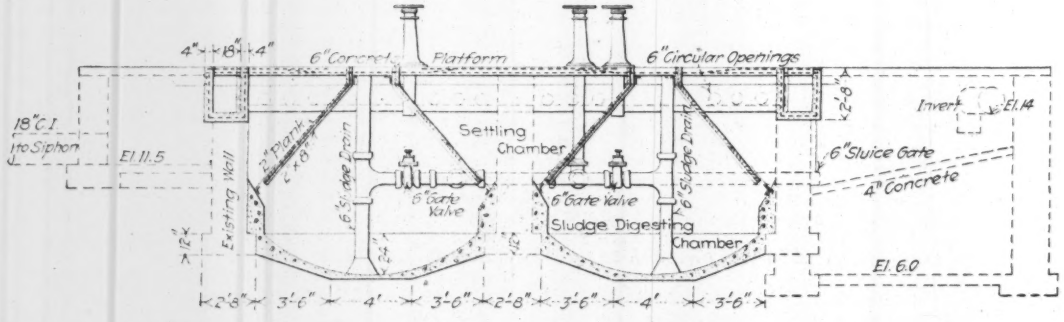


FIG. 1. CONVERTING A SEPTIC TANK INTO A TWO-STORY TANK, MOORESTOWN, N. J.

Fig. 2. The following notes on the tank are taken from Mr. Potter's paper:

The remodeled tank consists of four units, each of which is again divided into three compartments by a false bottom constructed of 2-in. creosoted yellow pine. This false bottom makes a slope of 40° with the vertical. The upper two compartments are settling compartments and the lower and larger compartment is the sludge-digestion compartment. The total settling capacity amounts to a retention of 1.35 hours when the plant is operated at the rate of 500,000 gal. daily. The total capacity of the sludge-digestion chamber figured below the level of the walls is 5470 cu.ft., about 50% larger than required by the established practice.

The system of distributing the sewage to the various units is of interest as it possesses not only flexibility of operation but also is very efficient in preventing disturbances from currents and vortex motion in the settling compartments.

After leaving the remodeled grit chambers, the sewage enters the main distributing trough, a concrete rectangular channel built directly upon the existing wall of the settling tank. This channel extends the entire length of the basin. Extending across the tanks are four lateral rectangular troughs, each 15 in. wide and 2 ft. deep. These lateral troughs can be used either as distributors or collectors. For this purpose each of the lateral troughs is provided with shear gates at either end so as to regulate the direction of the flow.

Attention is called to the method of leading the sewage into the settling compartment. The liquid enters the basin in the direction opposite to that it must take in traversing the basin. This method has proven to be very effective in arresting vortex motion by impinging the inflowing current against the wall of the basin. The collecting trough acts as a scum board and tends to prevent floating matter from being carried out with the effluent. In each unit the direction of flow can be reversed by the simple process of closing two shear gates and opening two others. Reversing the flow at periods of about a month tends to the more uniform distribution of the sludge in the sludge-digestion chamber.

Six transverse concrete walks, each 3 ft., wide, give the operator ready access to all parts of the tank. Except for these walks, the top of the tank is entirely open.

Wooden covers are provided for the main distributing channel and the main collecting channel so that during the winter months these channels can be covered to prevent the freezing of the sewage at the dead ends.

An 8-in. cast-iron pipe conveys the sludge from the tank to the sludge bed the surface of which is 5½ ft. below the water level in the settling tank.

Changing Contact Beds into Sprinkling Filters

To change contact beds with 4-ft. depth of medium to sprinkling filters of 5- to 5½-ft. depth, the bottom was excavated from 6 to 12 in. and the surface raised 6 in.

The funds being limited and the subsoil clayey, the filtering material was placed directly on the earth. For underdrainage there was provided a central concrete channel 15 in. in diameter, from which at right angles there were laid on 24-in. centers 6-in. vitrified bell-and-spigot pipe, with open joints. The central channel is ventilated at either end.

The sprinkler system was described in the paper as follows:

It was found that the maximum head available to operate the nozzles was about 4 ft. 9 in. A circular type of nozzle was used which was spaced 10⅓ ft. c. to c. in rows spaced 9¾ in. apart. To reduce the frictional losses in the siphon and piping system to the minimum, the siphon was made 24 in. in size—somewhat larger than customary. The main distributor and all piping were made of ample size so as to keep the frictional losses, even when the plant is extended in the future, within 6 in.

The nozzles were furnished by the Pacific Flush Tank Co., and are provided with adjustable orifices. Adjustable orifices, although not generally used except for experimental purposes, are of value in such a plant in that the operator can curtail the discharge from those nozzles over any portion of the bed where a tendency to pool and clog exists. This adjustable orifice has proved of value for when the plant had been in operation several months, an area about 10 ft. by 25 ft. in extent, over which the teams traveled in placing the filter material and which had been compacted by traffic, commenced to pool. This tendency was quickly remedied by throttling the nozzles which discharged over this area, thereby doing away with the tendency to pool.

The cost of remodeling the plant was as follows:

Remodeling septic tank into two-story tank	$3,100
Construction of dosing tank, siphon and piping	1,800
Sprinkling filter, ¼ of an acre	8,300
Sludge bed and piping	300
Additional piping around plant	1,300
Final settling basin	500
Total	$15,300

The unexpectedly large cost of converting the contact beds into sprinkling filters was due to the fact that less than 10% of the old filtering medium could be used in the new beds. Three of the original beds were composed of ¾ to 1½-in. slag and one bed was made up of cinders.

The effluent from the remodeled plant is clear, free from sediment, and of low turbidity, and "nonputrescible during the greater part of the time," Mr. Potter states. It now appears that the old "grit chamber" is entirely unnecessary and should have been abandoned when the plant was remodeled."

State Control of Water-Supply and Sewerage in Kansas

Under a law passed in 1907 permits must be secured from the State Board of Health to furnish water for domestic purposes and to discharge sewage into the waters of Kansas. Before such permits are granted, it is necessary to file with the board an application, report, plans and specifications for all proposed work. Permits are good for two years, but additional permits are required for extensions or alterations to plans.

There are now in Kansas 201 water-works, of which 40 supply surface water and 160 ground water. Of the 40 surface supplies 24 are subjected to mechanical filtration, and the remainder are treated in various ways. Three of the cities having the remaining plants voted bonds in December for filter plants and it is hoped that four more will soon take that step. The State Board of Health is conducting a vigorous campaign to secure filtration for all the unfiltered surface supplies. In many parts of the state water softening is much needed. Two softening plants have been under consideration, and one will be built in 1915.

There are 81 sewerage systems and about 50 sewage-disposal works in Kansas. In addition, two treatment plants are under construction and two will soon be built under orders from the State Board of Health.

Frequent analyses, for the most part weekly, are made of all the surface waters of the state. The cities buy their own containers for shipping samples, using the outfit described in Engineering News of Oct. 23, 1914, and also using alkalinity and turbidity kits, which are furnished by the State University at cost.

Twice a year tests are made of all water-filtration plants. Ground supplies are inspected and samples analyzed once a year or oftener. Sewage-disposal plants are inspected and tuned up twice a year, or oftener. The tests and inspections are made by the Division of Water and Sewage of the State Board of Health under the direction of C. A. Haskins, engineer, and with the aid of two assistant engineers, Fred R. Hesser and J. E. Welker. The laboratory of the State Board of Health has as its director C. C. Young. He has four assistants working full time and one assistant working part time.

Wind Stresses in Steel Mill-Buildings

By R. Fleming*

SYNOPSIS—Discusses the distribution of wind pressure on a sloping roof, referring to the experiments of Irminger, Kernot, Stanton, Smith and others. Analyses of stresses in Fink roof trusses show that a uniform vertical excess load is sufficient to take care of wind stresses if rigid members are used. In kneebraced mill-building bents, wind corrections are necessary. Suction effects are to be neglected except as regards anchorage. Recommends wind pressures and unit stresses, and discusses special bracing.

In designing ordinary mill-buildings it is common practice either (1) to neglect the wind stresses or (2) to calculate them in accordance with some textbook method and then tone down the results. In doing the latter, the general practice of designing buildings is followed, in conformity to which structures have been built that have rendered excellent service for many years. To bridge the gap between theory and practice, recourse is being had by some to what might be called a new school, which has advanced new methods and new experimental results. In the present article this school will be briefly reviewed, its conclusions negatived, and textbook assumptions made to agree as near as possible with actual conditions—the object being to present a safe, sane, workable method of determining and making provision for the wind stresses in steel mill-buildings.

Amount and Distribution of Wind Stresses

A recent writer[1] of the new school states the case thus:

> In a high wind the maximum pressure against the roof is at the windward eaves. The pressure decreases upward on the windward slope, and is zero, it is claimed, at a point three-fourths the distance to the ridge. Beyond the zero point, up to the ridge and down the leeward slope, the pressure is negative. The wind deflected upward by the windward surface of the roof rarefies the air over the leeward surface, which allows the air inside the building to exert an upward pressure in excess of the downward pressure on the roof. In other words, there is direct or inward pressure on the windward slope of the roof, center of pressure below middle of slope, and at ridge and on all of leeward slope, there is outward pressure or suction.

Suction on Roof

In 1894, J. O. V. Irminger, manager of the Copenhagen Gas Works, made a number of experiments on wind pressure, the description and results of which he embodied in a paper[2] to which reference is often made. A rectangular opening about 6½x11 in. was made in a chimney 5 ft. in diameter and 100 ft. high. Into this opening was inserted a conduit 4½x9 in., polished on the inside to reduce friction. Currents of air were made to strike plates and models placed in this conduit and the resultant pressure registered. A model of a pitched roof with 45° slopes showed a normal uplift on the leeward side due to suction three times as great as the normal pressure on the windward side. The conclusion drawn was "if the author's experiments on models represent the facts with regard to buildings, the methods with which roof principals are commonly calculated for wind-pressure need revision." An enthusiastic admirer of Irminger writes,[3] "It will be due to him that we surely in the future shall save tons of material in our roofs."

In 1891-94, Prof. W. C. Kernot, of the University of Melbourne, made the experiments connected with his name.[4] By means of a gas engine and propeller, he discharged a jet of air 12 in. by 10 in., placing into this jet the plates and models he wished to test. He concluded that the usual method of calculating wind stresses in roofs applied only to roofs supported by columns under which the air could blow freely. With roofs of a low pitch resting on walls having parapets, he found a tendency to an uplift.

In 1893 and later, T. E. Stanton, of the National Physical Laboratory, England, made the experiments which have become widely known from the papers he contributed to the Institution of Civil Engineers.[5] From observations on models of roofs the sides of which were 3 in. by 1 in. and sloped at 30°, 45° and 60°, placed in a current of air having velocities of 10.0, 13.6 and 16.8 miles per hour, he writes, "The experiments appear to indicate beyond question the importance of a consideration of a negative pressure on the leeward side of roofs." From later experiments on pressure boards 5x5 ft. to 10x10 ft., he found the coefficients of wind pressure to be as follows:

STANTON'S COEFFICIENTS k IN FORMULA $P_n = kV^2$

(a) Roof mounted on columns through which air can pass

	60°	45°	30°
Windward side	+0.0034	+0.0028	+0.0015
Leeward side		negligible	

(b) Roofs of buildings in which the pressure on the interior may be affected by the wind.

	60°	45°	30°
Windward side	+0.0034	+0.0028	+0.0015
Leeward side	—0.0032	—.....	—0.0022

This coefficient gives the normal pressure on roof surface in lb. per sq.ft., if V is the wind velocity in miles per hour, the wind blowing horizontal.

Prof. Albert Smith in a paper read before the Western Society of Engineers, November, 1910, entitled "Wind Loads on Mill Building Bents,"[6] among his conclusions advocates "placing the wind loads equally on the two walls, and inward and outward on the windward and leeward roofs respectively, as giving important changes of stress in members of the roof truss, as giving less stress in the kneebraces and columns, and as permitting the rational design of the girts." In 1912, he made a number of observations on a model building 6 ft. wide by 15 ft. long, with wall heights of 4, 5 and 6 ft. In a paper "Wind Pressure on Buildings,"[7] he writes:

*American Bridge Co., 30 Church St., New York City.

[1] "Insurance Engineering," August, 1912.

[2] "Engineering News," Feb. 14, 1895; "Engineering," Dec. 7, 1895; Proc. Inst. Civ. Engrs., Vol. CXVIII, p. 468.

[3] Theodore Nielsen, "Engineering," Oct. 9, 1903.

[4] "Engineering Record," Feb. 10, 1894; Proc. Inst. Civ. Engrs., Vol. CLXXI, p. 218; Australian Association for the Advancement of Science, Vol. V (1893), p. 573, Vol. VI (1895), p. 741.

[5] Proc. Inst. Civ. Engrs., Vol. CLVI, p. 78, Vol. CLXXI, p. 175.

[6] Journal Western Soc. Engrs., February, 1911.

[7] Journal Western Soc. Engrs., December, 1912.

The ordinary methods of assuming wind loads on mill buildings ought to be somewhat revised. For the case of roof trusses on masonry walls, or on steel bents with long diagonals, a suction effect in the neighborhood of 0.4 of the unit wind pressure should be placed on the leeward roof of all closed buildings, and a pressure or suction derived from the curves drawn from the observations placed on the windward roof. The resulting stresses will not only be different in amount from those computed on the present basis, but will in many members differ as to sign. Wind loads on purlins might in most cases be entirely omitted. * * * * In buildings with kneebraced bents, in addition to the preceding points, the suctions on the leeward wall should be considered.

Prof. Boardman, University of Nevada, in 1911 made experiments on a model roof 10 ft. long, each slope 6 ft. wide, resting on walls 4 ft. high. His conclusions are similar to those of Prof. Smith.[5]

An English textbook, Brightmore's *Structural Engineering,* first issued in 1908, quotes the Stanton experiments as authority and the stress diagrams for the roof truss given are made with the wind forces so acting. The heading of the section is significant: "Stresses Due to Wind Pressure and Wind Suction."

Another English textbook, Andrews' *The Theory and Design of Structures,* in an appendix to the last edition, 1913, calls attention to Stanton's conclusions and gives a stress diagram for a truss with the wind loading in accordance with these conclusions. In mentioning the stresses due to suction on the leeward side the author writes, "Few designers appear to have allowed for this in their calculations for roofs, but the question is of considerable importance and the results of these experiments should either be disproved, or allowance should be made for them in design."

Marburg in *Framed Structures and Girders* alludes to the experiments of Kernot, Irminger and Stanton and reproduces one of the Irminger sketches. His practical conclusion is:

The experiments of Kernot, Irminger and Stanton were made on much too small a scale to admit of quantitative deductions applicable to conditions in practice. They are valuably suggestive, however, in calling attention to conditions which were previously not generally or adequately recognized.

With this conclusion the writer is in thorough accord.

Selecting a Wind Pressure for Design

Our knowledge of wind pressures is very imperfect. It is generally agreed that the fundamental equation $P = kV^2$ is correct for horizontal wind. There is little dispute that for wind with a uniform velocity and normal to plates of moderate size, the value of k is from 0.0032 to 0.004. Of the formulas for wind pressure on inclined surfaces our best knowledge indicates that of Duchemin as the most accurate. It is

$$P_n = P \frac{2 \sin A}{1 + \sin^2 A}$$

There remains to be assigned a value to V. Average wind velocities for a day or a month or a year are useless. Shall the highest wind velocity on record be taken? Is this likely to occur again?

It is useless to attempt to make provision for tornadoes or violent hurricanes "against which neither care, nor strength, nor wisdom, can avail."* Such storms are limited in area and come but seldom, perhaps once in a century. The endeavor to make a mill-building strong enough to resist them would not only add greatly to the cost but would be ineffective.†

The highest wind velocity recorded in New York City since 1871 by the U. S. Weather Bureau was 96 miles per hour sustained for a period of five minutes. During one minute of that time the velocity was 120 miles per hour. This was in Feb. 22, 1912, a Robinson anemometer being used in the same location as at present, about 20 ft. above the roof of the 33-story building at 17 Battery Place. A recorded velocity of 80 to 90 miles is not uncommon. A recorded velocity of 90 miles per hour corrected by the Weather Bureau formula gives an actual velocity of 69.2 miles per hour. With this value in the formula $P = kV^2$, $k = 0.004$, the normal pressure is 19.2 lb. per square foot on a vertical surface. Wind pressure increases with the distance from the ground and decreases per square foot as the area becomes larger. When it is remembered that the instrument above mentioned is about 400 ft. from the ground and the cups are only 4 in. in diameter, the assumptions that will be made of a horizontal wind force of 20 lb. per sq.ft. in designing the trusses of mill-buildings, and 15 lb. in designing columns and kneebraces, seem to be ample and fully warranted.

Wind Stresses in Roof Trusses

Roof trusses resting on brick walls will first be considered. The example taken will be a roof truss as in Fig. 1, with pitch of 6 in. to 1 ft., and span of 60 ft. c. to c. of bearing plates. Trusses are 16 ft. apart on centers. For a horizontal wind from the left, with pressure of 20 lb. per sq.ft. on a vertical surface, the normal pressure on a surface inclined 6 in. to 1 ft. will be (by Duchemin's formula) 14.9 lb. per sq.ft.

The following cases will be considered:

(1) Wind load of 15 lb. per sq.ft. or 2012 lb. per panel, normal to one slope of roof, both ends of truss fixed.

(2) Loads as in (1), left end fixed, right end on rollers.

(3) Loads as in (1), left end rollers, right end fixed.

(4) Load of 15 lb. per sq.ft. exposed surface or 2012 lb. per panel on both sides of the roof, the loads applied vertically.

The stresses for these four cases are tabulated below:

It is seen at a glance that Case 4 is sufficient to cover wind stresses. The slight excess in a few members found in the other cases is negligible, especially when they are considered with the combined stresses due to all loads.

With the same wind velocity as before, according to Stanton, the pressure on the windward slope is about 7½ lb. per sq.ft. and $^{22}/_{15}$ times 7½ or 11 lb. per square foot negative pressure or suction on the leeward side. The forces acting upon the truss are as in Fig. 2 (reactions are for both ends fixed, wind shear equally divided), while Fig. 3 is the stress diagram for both ends fixed.

The tabulation of stresses given below is for

(5) Both ends of truss fixed.

(6) Left end fixed, right end on rollers.

(7) Left end rollers, right end fixed.

It might be stated here that all the above cases with

[5]Journal Western Soc. Engrs., April, 1912.

*This is the way it is stated in the Foreword of a little volume issued by the Home Insurance Co., of New York, advocating windstorm and tornado insurance. More than a hundred photographs in this volume of wrecks caused by windstorms illustrate the truth of the Foreword.

†Editor's Note—This argument in its terms applies just as much to office buildings and all other structures as it does to mill buildings. The author probably means to emphasize the cost limitation, for mill-buildings alone.

one end on rollers are hypothetical, as roof trusses under 100-ft. span are seldom built with other than fixed ends.*

RECOMMENDED DESIGN LOAD—Maximum wind loading comes seldom and lasts but a short time. The working stresses used for this loading may therefore be increased 50% above those used for ordinary live- and dead-loads. A wind load of 15 lb. per sq.ft. is thus equivalent to a load of 10 lb. using the working stresses for other loads.

The snow load varies from 20 lb. per sq.ft. horizontal projection in the latitude of New York City to 30 lb. in parts of New England. This is equivalent to 16.6 up to 25 lb. vertical load per sq.ft. surface of a 6-in. pitch roof.

For combined snow and wind a load of 25 lb. per sq.ft. over entire surface, acting vertically, is ample for roofs in the latitude of New York City. If to this is added the weight of trusses, purlins, and roof covering, reduced to square foot of exposed surface, we have the total load for which the ordinary roof truss should be designed. However, not less than 40 lb. should be used except in tropical climates with no snow, where the minimum loading should be 30 lb. Where snows are severe 5 to 10 lb. should be added to the 40 lb.

NO ALLOWANCE FOR SUCTION—Turning to the tabulation of stresses found by the Stanton assumptions, and taking into account the total stresses from all loads, the saving due to reduced wind stresses is small. A serious objection to taking advantage of even this saving is that with a monitor along the ridge, or openings in the building and roof, the closed roof may become a partly open roof, thus changing the conditions for which the assumptions were made. For a truss resting on brick walls the tendency to an uplift can be met by firmly anchoring at the ends. The tendency to reversals of stress can be sufficiently met by using stiff shapes for all members; flats and rounds have no place in an ordinary roof truss.

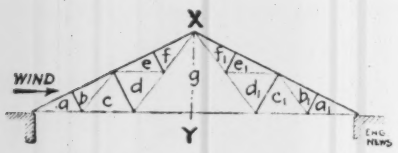
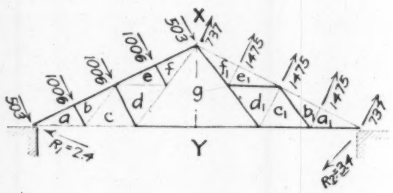


FIG. 1. SIMPLE ROOF TRUSS ANALYZED FOR WIND STRESSES



FIGS. 2 AND 3. ROOF TRUSS UNDER WIND PRESSURE AND SUCTION, WITH STRESS DIAGRAM FOR BOTH ENDS ANCHORED

STRESSES IN ROOF TRUSS FIG. 1, PRESSURE ONLY

Member Case	Xa	Xb	Xe	Xf	Ya	Yc	Yg	ab	bc	cd	de	ef	dg	fg
1	—9.1	—9.1	—9.1	—9.1	+10.2	+8.0	+3.5	—2.0	+2.3	—4.1	+2.3	—2.0	+4.5	+6.3
2	—9.1	—9.1	—9.1	—9.1	+11.4	+9.1	+4.6	—2.0	+2.3	—4.1	+2.3	—2.0	+4.5	+6.3
3	—9.1	—9.1	—9.1	—9.1	+7.7	+5.5	+1.0	—2.0	+2.3	—4.1	+2.3	—2.0	+4.5	+6.3
4	—15.8	—14.9	—14.0	—13.1	+14.1	+12.1	+8.1	—1.8	+2.0	—3.6	+2.0	—1.8	+4.1	+6.3

Member Case	Xa_1	Xb_1	Xe_1	Xf_1	Ya_1	Yc_1	a_1b_1	b_1c_1	c_1d_1	d_1e_1	e_1f_1	d_1g	f_1g
1	—5.1	—5.1	—5.1	—5.1	+3.5	+3.5	0	0	0	0	0	0	0
2	—5.1	—5.1	—5.1	—5.1	+4.6	+4.6	0	0	0	0	0	0	0
3	—5.1	—5.1	—5.1	—5.1	+1.0	+1.0	0	0	0	0	0	0	0
4	—15.8	—14.9	—14.0	—13.1	+14.1	+12.1	—1.8	+2.0	—3.6	+2.0	—1.8	+4.1	+6.3

Stresses are given in thousands of pounds. — Denotes compression. + Denotes tension

STRESSES IN ROOF TRUSS FIG. 2, PRESSURE AND SUCTION

Member Case	Xa	Xb	Xe	Xf	Ya	Yc	Yg	ab	bc	cd	de	ef	dg	fg
5	—0.8	—0.8	—0.8	—0.8	+2.7	+1.6	—0.7	—1.0	+1.1	—2.0	+1.1	—1.0	+2.2	+3.1
6	—0.8	—0.8	—0.8	—0.8	+4.9	+3.8	+1.6	—1.0	+1.1	—2.0	+1.1	—1.0	+2.2	+3.1
7	—0.8	—0.8	—0.8	—0.8	+0.5	—0.6	—2.9	—1.0	+1.1	—2.0	+1.1	—1.0	+2.2	+3.1

Member Case	Xa_1	Xb_1	Xe_1	Xf_1	Ya_1	Yc_1	a_1b_1	b_1c_1	c_1d_1	d_1e_1	e_1f_1	d_1g	f_1g
5	+4.1	+4.1	+4.1	+4.1	—5.6	—3.9	+1.5	—1.7	+2.9	—1.7	+1.5	—3.3	—5.0
6	+4.1	+4.1	+4.1	+4.1	—3.4	—1.8	+1.5	—1.7	+2.9	—1.7	+1.5	—3.3	—5.0
7	+4.1	+4.1	+4.1	+4.1	—7.8	—6.2	+1.5	—1.7	+2.9	—1.7	+1.5	—3.3	—5.0

Note—Stresses are given in thousands of pounds. — Denotes compression. + Denotes tension

The writer believes that the assumption of a total uniform load per square foot of exposed surface applied vertically at the panel points, with the same working stresses used throughout, is specially well adapted to the design of roof trusses.

WIND STRESSES IN KNEEBRACED BENTS

KNEEBRACED BENTS—The case of an intermediate transverse bent of a kneebraced mill-building will now be considered. The example taken will be that shown in Fig. 4; span 60 ft., roof pitch 6 in. to 1 ft., height 14 ft. to foot of kneebrace and 20 ft. to bottom chord. Trusses are 16 ft. apart c. to c.

The wind pressure will be taken at 15 lb. per sq.ft. perpendicular to the sides of the building and the corresponding normal component on the roof at 11.2 lb. (For buildings over 25 ft. to the eave line the normal component of a wind load of 20 lb., or 14.9 lb., would be used for the roof.) The columns are assumed partially fixed at the lower end, with the point of contraflexure at one-third the distance between the lower end and the foot of the kneebrace; the upper ends are considered supported. The wind shear is divided equally between the two columns. Fig. 5 is the stress diagram.

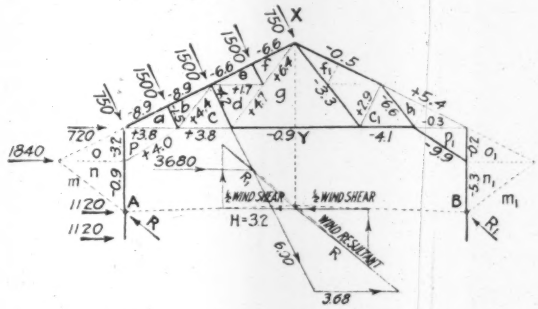
Bending moments in the columns are as follows:

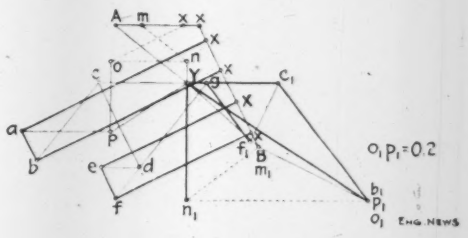
At the foot of windward column	12,320 ft.-lb.
At foot of leeward column	14,940 ft.-lb.
At foot of windward kneebrace	19,410 ft.-lb.
At foot of leeward kneebrace	29,870 ft.-lb.

It is seen that the maximum bending moment is at the foot of the leeward kneebrace.

*Editor's Note—The wind shear may, however, come wholly on one or the other wall, due to unequal bedding of the anchor bolts or to temperature movement. The condition then, as regards the present calculation, is identical with one end on rollers.

Recommended Method—For mill-building bents the writer first determines the stresses in the truss due to a total uniform load and then proportions it for the same. The ordinary working values for medium steel are generally used—16,000 lb. per sq.in. in net tension and (reduced by formula) for gross compression. If the wind stresses in any member from Figs. 4 and 5 are greater than the wind stresses from the uniform wind loading of 30 lb. per sq.ft. applied vertically, that member is proportioned for the maximum wind stress plus the stresses from the uniform loads other than wind, using working stresses 50% more than in the first calculation; but in no case is a less section used than that first obtained. The members bc and b_1c_1 will generally need be increased; often cd and c_1d_1; occasionally gd, gf, gf_1 and g_1d_1. A reversal of stresses is noted in certain members, particularly in bc and the lower chord. The diagonals bc and b_1c_1 can be made of two angles instead of one as when designed for tension alone. The compressive stresses in the lower chord are overcome by the tensile stresses due to the dead-load. The kneebraces have wind stresses only, and are proportioned for the larger working stresses.





Figs. 4 and 5. Kneebraced Bent of a Mill-Building; Stress Diagram for Wind Load
(Equal shears; contraflexure one-third height to kneebrace)

The column is first proportioned for carrying the direct stress due to the total uniform load from the truss, noting the flange area required. From the maximum bending moment due to the wind, as in Fig. 4 and 5, the sectional area required for the flanges is found using the larger working stresses and considering the column as a beam. If this flange area is not more than one-third of that first found no change is made; if more than one-third, the excess is added. The compressive stress due to overturning need not be considered unless it exceeds the stress from the wind portion of the uniform roof load.

Girts—The side and end girts are proportioned as beams supported at each end with a uniform horizontal load of 15 lb. per sq.ft.; an extreme fiber stress of 24,000 lb. per sq.in. is used.

The girts are apparently the simplest portion of the building to design, but if observation and experience count they are the most difficult. Side and end girts of $3\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$-in. or $\frac{5}{16}$-in. angles, 16 to 20 ft. long and 5 ft. apart, are still in use after 20 years' service, in defiance of all figures. Notwithstanding this, such designing practice is reprehensible.

Special Cases of Mill-Building Bracing

Traveling cranes running through a building often bar the use of kneebraces. The gussets connecting the trusses to the columns should then be as large as possible and calculations made accordingly. An ideal way is to transfer the transverse thrust from the wind as well as that from the cranes to the ends of the building by means of diagonal bracing in the plane of the lower chords of the trusses, and thence by diagonals in the ends of the building to the ground. Openings in the ends often interfere with this expedient. Neither is it feasible in building so long as to require provision for longitudinal expansion and contraction due to changes in temperature.

In all cases diagonal bracing should be introduced into the planes of both top and bottom chords for stiffness as well as to take calculated stresses. This applies also to roof trusses resting on brick walls. Adjustable rods can be used for top-chord bracing, but the bracing of the bottom chord should be entirely of angles or other rigid shapes, with bolted or riveted connections.

The ends of a building, the gables in particular, are more liable to be severely strained from wind than any other portion of the building. Generally diagonals in the planes of the chords of each end panel, and in each end side bay to the ground will be sufficient to take care of the induced stresses. If not, the shear may be divided with other braced bays.

Special types of buildings should be considered in reference to their own requirements. Open sheds, especially if the gables are closed, may have an uplift as well as a vertical load.

Comments on Prior Recommendations

The leading textbook on the subject of mill buildings is Ketchum's *Steel Mill-Buildings*. In this book a kneebraced mill bent is considered for four cases:

(1) A horizontal wind load of 20 lb. per sq.ft. on the side and vertical projection of the roof, with the columns hinged at the base.

(2) Same wind load as in Case 1, with columns fixed at the base.

(3) A horizontal wind load of 20 lb. per sq.ft. on the side, and the normal component of a horizontal wind load of 30 lb. per sq.ft. on the roof, with columns hinged at the base.

(4) Same wind load as in Case 3, with columns fixed at the base.

The writer believes that the loads of 20 and 30 lb. are larger than need be used. The columns are seldom if ever rigidly fixed at the base, neither are they hinged. That they are partially fixed and the point of contraflexure is at one-third the distance from base to foot of kneebrace is believed to be nearer correct than either assuming it one-half the distance or assuming the columns supported at the base. There seems no good reason for assuming a normal component of a horizontal wind load

of 30 lb. on the roof while a horizontal wind load of 20 lb. is taken on the sides. It is not clear why the Hutton formula is used to find the normal component. Near the beginning of the book we read, "Hutton's formula is based on experiments which were very crude and probably erroneous. Duchemin's formula is based on very careful experiments and is now considered the most reliable formula in use." The specifications near the close of the book call for the Duchemin formula to be used in computing the normal wind pressure; by this formula, for a 30-lb. load, the 18-lb. normal pressure in Cases 3 and 4 would be 22.4 lb. The only increase of the usual working stresses allowed is 25% for laterals and 50% for combined direct and flexural stress due to wind. This increase does not apply to the combination of wind with other loads though with the maximum wind load a minimum snow load of 10 lb. per sq.ft. is allowed. (For the purpose of aiding those who wish to make comparisons the same roof truss and bent have been taken in this article as found in Ketchum.)

The chapter on "Iron and Steel Mill-Building Construction" by G. H. Hutchinson in Johnson, Bryan & Turneaure's *Modern Framed Structures* considers three cases of a mill-building bent, arriving at conclusions similar to those of Ketchum. The method of obtaining reactions and moments is quite abstruse and difficult to follow. No mention is made in the chapter of working stresses to be used.

Smith, in "Wind Loads on Mill-Building Bents,"[6] assumes a total horizontal wind force of 30 lb. per sq.ft. on the bent considered in this article. The pressure on the sides is divided equally between the two columns. One-third of the normal component of the total pressure on the roof, found by Duchemin's formula, is taken by the windward slope and two-thirds as suction or an outward pressure on the leeward slope. The bases of the columns are considered hinged. Comparing his results with those obtained by Ketchum, he shows reduced stresses but is unfortunate in the example selected for comparison: Ketchum's stresses are taken and 50% is added to them for a 30-lb. load; but Ketchum calculates his roof for the normal component of a horizontal wind of 30 lb. and the side for 20 lb., so that Smith is actually comparing his results with those obtained from a bent having a pressure on the sides of 30 lb. per sq.ft. and on the roof the normal component (by the Hutton formula) of a horizontal force of 45 lb. per sq.ft.

However, with the same wind force, the Smith method does give reduced stresses, especially in the columns, kneebraces and girts. The important point is whether these reductions are permissible. In a modern mill-building the sides are from one-fourth to one-half or more of glass, a large proportion of which can be opened to permit of ventilation. If opened on one side only, Smith's assumption that the pressure on the inside of a mill-building is a mean between the windward pressure and the leeward suction disappears. In a high wind the windows on the leeward side are liable to be open and those on the windward side closed; there is then little suction. In a building the sides of which are covered with sheet metal there is always a probability of the covering on one side or end being removed for 8 ft. or more from the ground, thus completely doing away with the suction theory. For these reasons it is unwise to take advantage of a theory based upon assumptions which are destroyed by a probable change of conditions.

In conclusion, while the methods advocated for treating wind stresses may not be thoroughly scientific, *they are easily workable, and experience proves that they are safe and sane.* The load of 15 lb., the working stress of 24,000 lb., and the assumed point of contraflexure, may all be criticized, but for the ordinary mill-building it is more rational to use these assumptions and make strict provision for them than to follow the present method of giving an intellectual assent to the theories of the text book and ignoring them in actual practice.

Some Experiences with Concrete in the Republic of Panama

ALEX P. CRARY*

Because of the rapid deterioration of both steel and wood due to the climatic conditions and to the neglect of the Panama Government in maintaining structures, the late Ricardo M. Arango, Chief Engineer of the Republic, decided that so far as possible all structures should be built of concrete. This decision was further favored by the fact that frost, one of the most destructive forces against concrete, is totally absent in this country. Some experiences with the structures built as a result of this decision should interest engineers.

EFFECT OF SEA WATER ON CONCRETE

The first structures undertaken were a wharf and a public market in the City of Panama. The wharf and part of the market are supported upon concrete piers which are so located that the greater part of each pier is between high and low tides, which is the region in which sea water exerts its most destructive action, if it exerts any at all. The location of these piers in respect to the sea is best shown in Fig. 1, which was taken at low tide; the high tide often goes within one foot of the cross-girders. At the time these piers were being placed, the fall of 1911, there was considerable discussion as to the effect of sea water on concrete. In view of this, the ingredients of which the concrete for different piers was composed was varied and records of the same kept. This variation extended to the stone and sand as well as to the cement, to the proportions and consistencies of the mixtures and to the use of a waterproofing compound.

All of the sand was taken from sea beaches. The stone was of varying sizes. The chemical compositions of two of three brands of cement used are as follows:

	No. 1	No. 2
Loss on ignition	7.40%	1.55%
Silica dioxide (SiO_2)	23.67	22.44
Iron oxide (Fe_2O_3)	6.65	4.16
Aluminum oxide (Al_2O_3)	1.80	7.24
Calcium oxide (CaO)	57.88	61.82
Magnesium oxide (MgO)	1.28	1.18
Sulphur trioxide (SO_3)	1.33	1.82

No. 2 was a German cement and No. 1 an American. Neither of the cements analyzed was in first-class condition, due to long storage and to breakage.

In the mixtures different combinations of large and small stones were used. The proportions varied from 1:3:6 to 1:4⅘:12. The lean mixtures were due to a mistake which was not noticed until after a number of

*Box 23, Ancon, C. Z., Panama.

piers had been poured. The consistency varied from a wet, not sloppy, mixture to a comparatively dry one in which the concrete would just quake when rammed.

In most of the piers the concrete was laid without a waterproofing compound. But in a few a waterproofing grout was poured in against the face at the time of placing the concrete. This waterproofing compound was made up of 2 cu.ft. of cement, 4 cu.ft. of sand, 3 quarts of stock solution and water enough to make a grout. The stock solution was mixed in the proportion of 2 lb. caustic potash, 5 lb. powdered alum and 10 quarts of water.

In constructing the piers the excavation was made, the forms placed and the concrete poured, all for one pier, during one low-tide period. In most cases the rising tide would follow closely the upper surface of the concrete as it was brought up. All the concrete was thoroughly mixed in a batch mixer.

After three years' exposure to the sea water, from the corners of each pier tested there were broken off, with

FIG. 1. LOW TIDE AT WHARF, PANAMA, SHOWING FOUNDATION PILES

hammer and chisel, two pieces of concrete. The piers then had a light covering of marine animal and vegetable growth. One piece was taken from the lower part which is submerged more than one-half of the time, and the other was taken from the part which is above high tide and, therefore, which is at no time in contact with sea water. Pieces were also taken from some piers which had been previously built and which had been exposed to the action of sea water for at least five years. The pieces were tested by breaking off the edges with the hands, by the feel and by the appearance.

In all cases without exception, the breaking off of the edges with the hands and the feel revealed no difference in the strength between the two pieces taken from the upper and lower parts of the same pier. The only difference that could be detected was that, in the piers made of lean concrete, the pieces taken from the lower parts were saturated, while those taken from the upper parts were dry. If sea water has a deteriorating effect on the concrete of the piers of the Panama wharf and market, whatever the mixture or combination of ingredients used, it will take more refined methods of testing than those described above to detect it.

The fact that the pieces of lean concrete which were saturated showed up as well as the pieces not subject to the action of sea water but taken from the same piers would seem to indicate that sea water alone does not affect concrete. Had these piers been located in regions where moderately severe cold weather prevails at times, the saturated parts of the lean concretes would probably have disintegrated.

As to the contention that a cement high in iron oxide and low in aluminum oxide is best for use in the sea, it is to be observed that the No. 2 cement, which was used in many of the piers and which had the usual amount of aluminum oxide, did not show any inferiority.

Several of the reinforced-concrete beams of the wharf had to be placed so low that at extreme high tides they are in part submerged. In all cases where good concrete covers the reinforcing steel it is properly protected. A piece of No. 14 annealed wire which was not more than skin deep in the concrete was found, upon being cut out, to be as good as when it was placed three years previous.

REINFORCED-CONCRETE ROOFS

When the designs for the above mentioned wharf and market were complete and the estimates made, it was found that sufficient money had not been appropriated to complete the structures as designed. To effect a saving the engineers decided to attempt so to construct the roofs that the concrete alone would be water-tight, thus doing without the usual felt roofing and its covering of gravel or of mortar.

The roofs were designed by the writer following closely to the usual practice. The slab reinforcing was loose bars designed to be laid straight in the bottom with none bending up over supports. When it was decided to build the roofs without waterproofing, it became evident that something had to be done to control the location of the contraction cracks which were bound to occur, and to make them water-tight. In roof work these generally coincide with the joints which mark the beginning and the ending of a day's work or of a single stretch of concrete placed in one operation. Therefore, the provision for making them water-tight was located at the joints in the middle of the slabs midway between the beams, where work is generally begun and ended. This provision consisted in turning up, as it were, the concrete of the slabs at the joints, thus making very shallow and wide troughs which sloped toward the edges of the roof. Then to make water-tight the joint between two adjacent turned-up edges, it was capped with concrete. This is best illustrated in Fig. 2.

In accordance with this scheme the roof work was started. After the first forms were removed it was found that, although none of the caps had been placed over the joints, the leaks occurred not at them but mostly at the beams. This indicated one of two things; either the cracks at the beams were due to the unequal setting of the concrete caused by its greater mass along them or the concrete was unable to resist the negative bending due to the absence of reinforcing in the tops of the slabs over them. That the joints did not leak to any considerable extent indicated that the slabs must have deflected, thus producing a tendency to close them.

This experience showed that, if we were to succeed in making the roofs water-tight, we would have to change our methods. Evidently we were on the right track so

far as the design of the joint was concerned; the trouble was at the beams. To proceed on the assumption that failure here was due to the unequal setting of the concrete and pour the beams and slabs separately, was out of the question. All beams were designed as T-beams which required that the slabs be poured with the beams. Therefore, in order to be able to pour them separately, a change in the design would have had to have been made. To do this was impracticable as the work had proceeded too far in accordance with the original plans. This left only the alternative of resisting the negative bending by placing reinforcing in the tops of the slabs over the beams. This was done and apparently with success. Besides this change the form of the joint was changed to that shown in Fig. 2, which was suggested by Howard Gibbons, the construction superintendent, and which

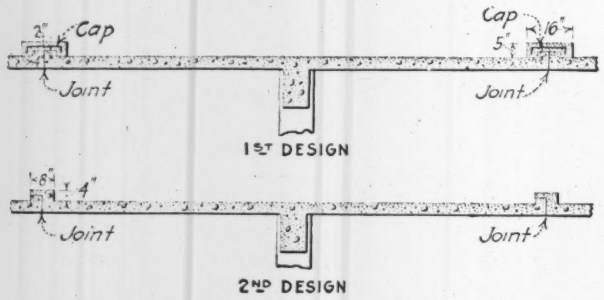


FIG. 2. DESIGNS OF WHARF ROOF, PANAMA

proved to be successful. The caps over the joints are shown in the view in Fig. 3.

The concrete was mixed in the proportion of 1:2:4, medium wet and was kept damp for at least a week after placing. The amount of water used for tempering it was no greater than that required to work it properly around the reinforcing. However, the concrete in one of the best sections of roof laid was exceedingly sloppy because of having been poured during a very heavy rainstorm. It was brought as near as possible to the elevation of the top without allowing stones to project above the finished surface. Within an hour after placing, a thin coating of 1:2 mortar was spread over it and troweled to a smooth but not slippery surface. The guides for the screed were supported by small blocks which rested on the slab forms. In some cases these were left buried in the concrete too long with the result that at these points the roof leaked. Leaks were repaired by pouring hot pitch into the cracks. During the hot, dry season the roofs shrink, causing the cracks to open up so that at the first rain of the wet season they leak. But as the concrete becomes more or less saturated with water they swell so that it is not long after the beginning of the rainy season before they are again practically tight.

It cannot be said that these roofs are a success in the full sense of the word though what few leaks do occur do no harm. It should be remembered, however, that these roofs were not originally designed with the view of making them water-tight by means of concrete alone, nor had the engineers seen anything in the engineering literature to guide them.

The experience gained leads the writer to believe that concrete roofs that will be absolutely water-tight can be constructed. To do this the slabs must be designed with that purpose in view; they must have sufficient negative reinforcing over the supports to resist the negative bending and ample steel to distribute the stresses caused by expansion and contraction; the concrete must be properly laid, having the top coat of mortar placed soon enough so that it will bond with the main body; and, if guides become buried in the concrete, they must be pulled just as soon as it is possible and the holes filled with mortar.

SEA WATER FOR TEMPERING CONCRETE

At Puerto Obaldia on the Pacific Coast there has been constructed a small wharf which is simply a reinforced-concrete floor resting on precast reinforced-concrete piles 30 to 36 ft. in length. These had been made a little over a year before being driven. In driving they were sus-

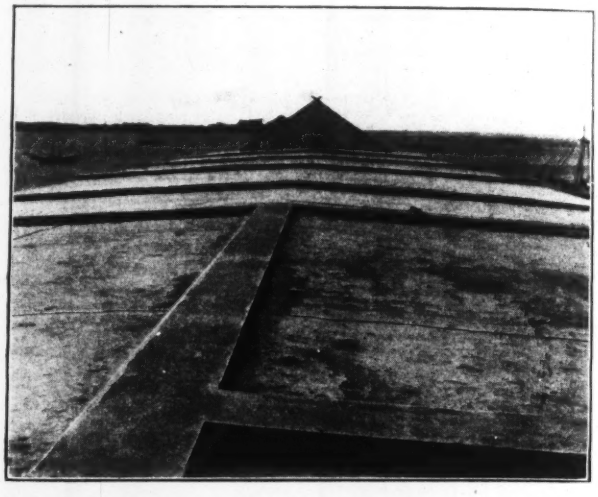


FIG. 3. ROOF OF WHARF, SHOWING CAPS OVER JOINTS

pended in place by a derrick and sunk by a water-jet forced through 1½-in. pipes placed in the centers of the piles. These piles rested in a thick deposit of sand which was overtopped by about 6 ft. of alluvial mud. At this port it was very difficult to procure fresh water, and all of the concrete was tempered with sea water. As the sand and gravel were taken from the sea beaches, everything in the wharf proper except the cement and the reinforcement contained salt. How this unusual amount of salt will affect the wharf in years to come, it will be of interest to learn. One year after the piles were made an examination of the reinforcement by cutting away some of the concrete revealed no injurious effects. The piles were well made, the concrete being well mixed and placed, free from voids and very dense.

Hints on the Construction of Slag-Tar Macadam Roads—During the past seven years I have laid many miles of slag-tar macadam roads for the Darlington Rural District Council. My first specification for carrying out this class of work was to use 2¼-in. bottom coating and 1¼-in. finishing coating, with a very thin sprinkling of clean ⅜-in. untarred slag. the result being that, although the road appeared to be rather open for the first few months after being laid, it eventually faced up to have the appearance of a mosaic pavement. I found, however, that I could get equally as good results and a road more suitable for the traffic of a rural district by putting down the road with material consisting of 70% of 2¼-in. and 30% of 1¼-in. mixed together and laid in one coat. In my opinion a great mistake made by road surveyors in laying a tarred-slag road is the use of a fine tarred topping material. Almost invariably this topping will eventually come off in places.—John Robinson, Surveyor to the Darlington Rural District Council, in the "Surveyor," Nov. 20, 1914.

Dry Sand and Cement Mixture vs. Mortar Bed for Wood Block Pavements*

By Theodor S. Oxholm†

In this country it has been the custom for many years to lay wood-block pavement on a concrete base with a cushion of sand or a bed of mortar between the base and the blocks. At present in European cities the concrete base is laid perfectly smooth, and the blocks are placed directly thereon, though 15 years ago a mortar bed was used in London.

In order that American practice may be discussed with understanding, the specifications used in the Borough of Richmond, City of New York, follow:

On the surface of the concrete shall be placed a bed of [sand] dry mortar, composed of 1 part of cement to 4 parts of sand, ½ in. thick.

On the surface of the mortar bed shall be set strips of wood 3 in. wide by ¼ in. thick, or strips of steel of the same width by not less than ⅛ in. thick, and of the greatest length convenient for handling. These strips shall be carefully set parallel and about 8 or 10 ft. apart, running from curb to curb, and be imbedded in dry mortar throughout their lengths, so that the top surface shall be the required depth below and parallel to the grade of the finished pavement. The space between two strips having been filled with dry mortar, a true and even surface shall be struck by using an ironshod straight-edge on the strips as guides, and as soon as the bed has been struck, the strips which would interfere with laying the blocks shall be removed and their places carefully filled with dry mortar.

On this dry mortar surface, spread and smoothed as above to the proper crown and grade, the blocks are to be laid with the grain vertical, with close points and uniform top surfaces, in courses at right angles to the line of the street, except in and between car tracks, in intersections and in other special cases, when they shall be laid diagonally, as shall be directed.

When laid, the blocks shall be covered with clean, fine sand, entirely free from loam or earthy matter, perfectly dry and screened through a sieve having not less than 20 meshes per lin.in., the sand to be swept and brushed into the joints. The pavement shall then be rolled with a 4- or 5-ton roller, and sand spread over the pavement and left on the surface until such time, when, if required by the engineer, the pavement shall be swept clean for final inspection, and any defects then noted shall be remedied.

A sand cushion is intended primarily to smooth out the roughness and inequalities in the concrete, so that the blocks rest evenly thereon. Secondly, the yielding surface of the sand permits the roller to press the blocks into it until they present a smooth surface adjusting the slight inequalities in the depth of the blocks; and thirdly, the sand has a slight resiliency and protects the blocks somewhat from surface wear. The mortar bed performs the same office as an equalizer of the concrete surface and the surface of the finished pavement, but there the similarity ceases; for as the mortar gradually sets, it forms a hard unyielding bed for the blocks to rest upon, sacrificing resiliency for immobility.

There are two objections in the writer's opinion to the use of a sand cushion. First, when cuts are made for any purpose through the pavement, it frequently happens that weeks and months elapse before repairs are made; during this time, storm water works its way between the blocks and base and disturbs considerable quantities of pavement, that will have to be relaid. This is especially noticeable on streets with a considerable grade, and could not occur with a well set mortar bed. Second, it would seem that even the slight resiliency of the sand cushion means the unstable condition of each block with respect to its neighbors, and a consequent lack of support on the sides and ends, which is of the utmost importance.

The one objection to a mortar bed has always been that the mortar has been mixed damp, and time must be allowed for it to set hard (three or four days), before traffic can be admitted, whereas wood-block pavement on a sand cushion can be thrown open for traffic as soon as completed. The writer has overcome this objection by mixing the mortar dry, and allowing it to set as moisture reaches it through the joints which are always of sand. The roller and immediate traffic work the blocks down to their final beds before the mortar sets. Work of this kind has been examined at cuts, and it has been found that the mortar was set up hard, though traffic had been allowed on the new pavement as soon as completed, and the surface was still uniform.

Asphalt blocks have for many years been laid on a mortar bed, and this method has evidently been adapted for use in laying wood-block pavement. Some six years ago the writer used a damp mortar bed. One day the roller broke down at a time when a large yardage of block had been laid and was ready for rolling. Before the roller was repaired the mortar had set and the pavement on this section was never as perfect as on those adjoining. For this reason a sand cushion was substituted for a mortar bed in the specifications, until last year when dry mortar was permanently adopted. In the writer's opinion mortar is superior to sand for a bed for wood-block pavement.

[This practice corresponds closely to that at Kansas City, Mo., described in our issue of Nov. 5, 1914, by Clark R. Mandigo, Assistant City Engineer, except that the Kansas City specifications require the blocks to be thoroughly wet, after rolling but before the joints are filled, which insures sufficient moisture reaching the mortar bed to cause it to set before traffic comes on the pavement. This method has given eminent satisfaction.—Editor.]

*Paper read at the recent annual meeeting of the American Association for the Advancement of Science, Philadelphia.

†Engineer-in-charge of the Bureau of Engineering, Borough of Richmond, New York City.

Salaries of New York City Engineers a Generation Ago—The proposed standardization of engineering positions and salaries in the civil service of New York City, described and commented upon in "Engineering News," of Jan. 14, 1915, is of considerable moment to the engineering profession generally as indicative of the value of engineering services to our largest and most important city, and thence to any city in proportion. It is therefore both interesting and instructive to compare the present and proposed standard salaries (p. 54, Jan. 14, 1915) with the salaries of similar positions in 1887 ("Engineering News," Feb. 5, 1887). At that time the Commissioner of Public Works received $8000 per annum; a deputy commissioner $5500; a chief engineer $7000; a consulting engineer $4000, and nine assistant engineers $1800 to $3000. Levelers got $1200 to $1400; rodmen $1000 and $1200 and chainmen $900. Draftsmen received $1000 to $2000 per annum. The engineer in charge of sewers received $4000 per annum; four assistant engineers $2250 to $2700; rodmen $1200 to $1500 and draftsmen the same. The Department of Parks had two engineers of construction at $3000, a topographical engineer at $3000, 15 assistant engineers at $1200 to $2200 and draftsmen, computors, chainmen, etc., at $1200 per annum. The Department of Docks had a chief engineer at $6000 per annum, an assistant engineer at $3000 and one at $2000; draftsmen $900 to $1320. The chief engineer of the Aqueduct Commission, which corresponds with the present Board of Water Supply, received $10,000 per annum, a consulting engineer $5000; two principal assistant engineers $4800; a special assistant $5000; five division engineers $3000, two assistant engineers $2400; 18 assistant engineers $1800, draftsmen $1200 to $1500; transitmen, $1500; levelers, $1200; rodmen, $900; chainmen and axemen, $720. There were 170 engineering employees, all told. These figures show that the salaries of the higher position have advanced 50% to 100% in 28 years, while those of the lower ones have remained practically unchanged.

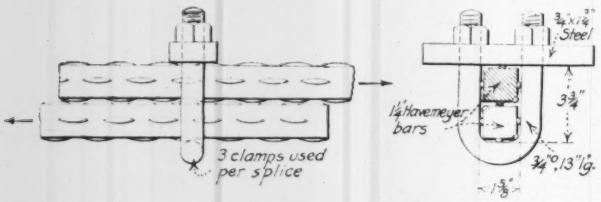
Field and Office

U-Bolt Splice in Heavy Reinforcing Bars

By Thomas C. Atwood*

During the construction of the Yale Bowl a very high reinforced-concrete retaining-wall was built, requiring the use of 1¼-in. square reinforcing bars. The retaining wall is of the inverted-T type, without counterforts. The foundation was put in first, the vertical leg being placed afterwards.

The main reinforcement bars were set in the foundation and projected 5 ft. above, giving a 5-ft. lap in the bars when the upper part of the wall was placed. In the highest part of the wall, which was 42 ft. above the top of the base, these bars were 3 in. on centers, leaving a comparatively small amount of concrete between them to carry the stress from one part of the bar to the other, and



U-Bolt Splice for Reinforcing Bars, as Used in Retaining-Walls of Yale Bowl

hence it was decided to strengthen the splice by the use of clamps. Some experiments were made to find what clamp should be used, and how many. For this size of bar the best type of clamp seemed to be that ordinarily used in fastening together wire cables, namely, the heavy steel U-bolt with crosshead of steel or cast iron.

Tests of Splicing Strength

In the first test a U-bolt of ¾-in. steel was used with heavy cast-iron crosshead. The reinforcing bars used were the Havemeyer bars, and the clamp was put over two of these bars and tightened up moderately. The first slip was at 1000 lb., and amounted to ¼-in. before holding again. The bars were then taken out and put into a vise and the clamp tightened up hard with a 24-in. wrench. This time the first slip was at 5100 lb., and the test was continued until the joint held 19,800 lb., by which time it had slipped 1¼ in., most of this, however, being after 15,000 lb.

The next test was on a ⅝-in. U-bolt with a plain steel strap crosshead ½ in. thick and 1½ in. wide. This was tightened up with the 24-in. wrench. The first slip was at 2700 lb.

The next test, on the same, showed first slip at 3200 lb.

For the next test *both clamps* were put on and tightened up hard. First slip occurred at 10,000 lb.

In the next test the two clamps were used again and put 16 in. apart, a ½-in. bar being placed between the reinforced bars halfway between the clamps, with the idea that by bending the reinforcing bars around this small bar a greater resistance to slipping might be obtained. This proved not to be the case, however, as the first slip occurred at 6200 lb.

After these tests were completed we attempted to break one of the reinforcing bars, but without success, the capacity of the testing machine being only 100,000 lb.; the maximum load on the bar was 98,000 lb. The yield-point apparently was at about 86,000 lb.

Three Clamps per Splice Adopted

It was decided to use three of the large clamps on each splice, to develop about 15,000 lb., which is about two-thirds the working load on the bars, thus relieving the concrete largely of stress and preventing entirely any tendency to split off the face of the concrete.

The placing of the clamps was carefully watched by the inspector, the nuts being tightened up by a 24-in. wrench. The crossheads used were ¾x1¾-in. steel bars, as these were found to be less expensive than the cast iron and apparently about as effective. The U-bolts were of ¾-in. steel with 1⅝x3¾-in. clear opening (bar 13 in. long), 2½-in. thread. A clamp complete weighed nearly 3½ lb. and cost 25c.

The tests were made in the Mason Laboratory of the Sheffield Scientific School by Dr. W. K. Shepard and the writer.

*Supervising Engineer, Yale Committee of Twenty-one, New Haven, Conn.

Pulling Steel Sheetpiles with an Inverted Steam Hammer

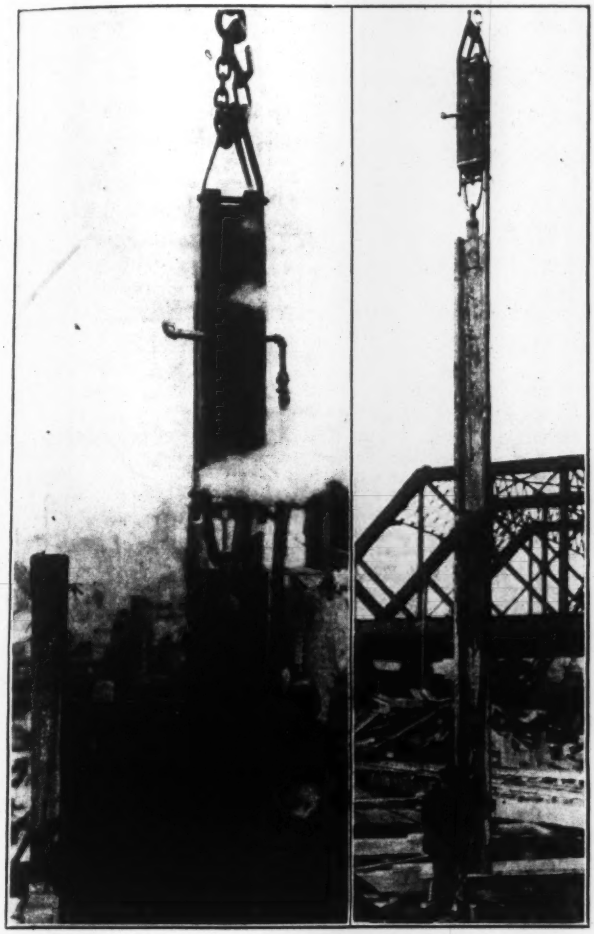
A steam pile-hammer fitted on the job with a few straps, bolts and a shackle, and hung upside down from a derrick boom, is being used to pull deep-driven steel sheetpiles at the Harlem River crossing of the new Lexington Ave. subway, New York City. On this job a large amount of sheetpiling is used in coffer-dam work in the construction of the land connections or approaches to the tubes. This piling was too costly to leave in place permanently. The familiar lever-and-derrick method of pile pulling was employed until about three months ago, when N. R. Melvin, superintendent for McMullen & Hoff, contractors on the section, arranged a No. 5 McKiernan-Terry pile-hammer for pulling.

A steel strap about 4 in. wide was passed entirely around the hammer over the anvil blocks and fitted with a simple shackle at the upper end (lower end in pulling). The 3½-in. space on each side of the strap was occupied by other straps to provide attachment to a derrick (see Fig. 1).

In pulling, the inverted hammer takes hold of the pile by means of a shackle, as illustrated. The derrick is then caused to pull steadily upward, which pulls the center or shackle strap down upon the anvil block, and the hammer is started working. The hammer used, 11x11x56 in., weighs 1500 lb., has 7-in. cylinder bore and stroke, and is

fitted with a 200-lb. ram. Its normal speed is 300 strokes per min.

On the northern side of the river it was desired to remove a double line of piling, penetration about 50 ft.



FIGS. 1 AND 2. INVERTED HAMMER PULLING SHEETPILING, NEW LEXINGTON AVE. SUBWAY, NEW YORK

(Left—Showing ordinary method of attaching inverted hammer to pile. Right—Lower length of pile pulled; shackle bolted to a pair of special plates which bolt to pile by means of its splice-plate holes.)

On account of the distortion of the piling due to boulders, etc., the bolt-hole at the shackle connection tore out frequently, in attempts to pull the upper length. For this reason the upper line was dug out and about 2 ft. of the lower line exposed. The lower piles were then withdrawn by the inverted hammer. Fig. 2 shows the shackle bolted to a pair of plates which are fastened to a pile by means of the splicing holes.

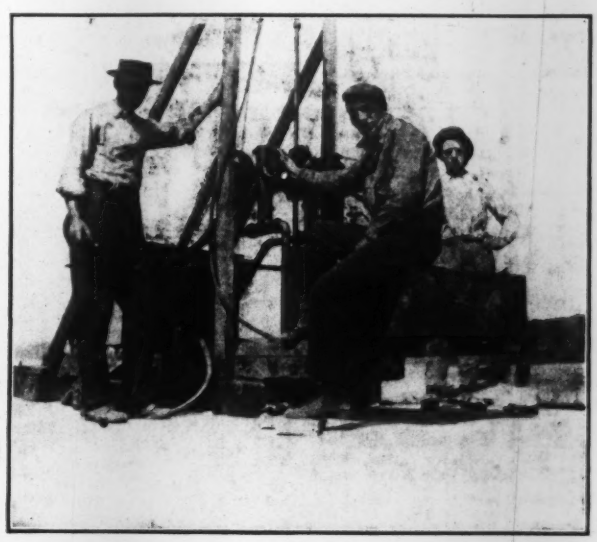
On this job piles penetrating 26 ft. in clay, sand and gravel ("ordinary hard pulling") were withdrawn in from 2 to 10 min. each when there was no distortion.

Cost of Drilling Anchor-Bolt Holes for the Quebec Bridge

BY C. C. PHELPS*

In constructing the new Quebec Bridge across the St. Lawrence River, 176 holes for the anchor bolts, each 4¾ in. in diameter, were drilled into the main piers. These piers are constructed of large blocks of granite as shown by the photographs. Each of the holes was bored to a depth of 5 ft., a total of 880 ft. A Calyx core drill, class G-O (Ingersoll-Rand Co.), taking out a 3⅜-in. core, was used for the work. Although this machine is rated for holes up to 3¼ in. only, it drilled the 4¾-in. holes with little difficulty.

The cost data tabulated below were furnished by George F. Porter, Engineer of Construction for the contractors, the St. Lawrence Bridge Co., Ltd. The cost includes all items connected with the drilling of the anchor-bolt holes except power consumed; but the latter can be roughly



DRILLERS AT WORK ON MAIN PIER OF QUEBEC BRIDGE

estimated, as a 5-hp. alternating-current motor was employed for operating the drill.

COST OF DRILLING 176 ANCHOR BOLT HOLES, DEPTH 5 FT.

Labor—		
North Shore, 2 drill runners	$501.85	
South Shore, 2 drill runners	577.20	
		$1079.05
Additional labor (getting started on North Shore)		140.40
Traveling expenses		40.22
2000 lb. steel shot		100.00
Expressage on shot		3.00
28 ft. of 3-in. belting and rivets		11.29
1 drill spindle, 4 ft. by 6 in.		22.85
1 shot feed complete		4.50
18 shot bits, 4½ in.		117.00
2—1-in. stop-cocks		1.86
1—4-in. core barrel plug		15.50
2—4-in. by 2-ft. core barrels		16.00
Total for 880 ft.		$1551.67
Cost per foot of hole		$1.76

The speed of drilling ranged between two and three holes per 10-hr. shift. Of course, the first few inches of each hole are always drilled at a much slower speed than after the drill has penetrated some distance into the rock. Considerable expense was involved in this case in conveying the material from some distance to the bridge site, for which purpose small boats were employed. Taking all these factors, as well as the low power consumption, into consideration, and adding a proper charge covering interest and depreciation on the drill (which cost $450), the cost per foot of hole drilled is remarkably low, in fact it is only a little over half of the contract price at which this work was let, namely, $3.50 per ft.

*Ingersoll-Rand Co., 11 Broadway, New York City.

Electric Suction Dredge with Special Cutter for Gumbo

The Cuyahoga River cutoff in the southern part of Cleveland, Ohio, which has been under way for two seasons (see illustrated article in ENGINEERING NEWS, July 2, 1914) is being dug by a suction dredge operated by electricity conveyed from the bank by cable. H. Fies, of the Marion Steam Shovel Co., Marion, Ohio, builders of the dredge, has furnished the following descriptive notes on this novel machine. It is understood that changes in the cutter retarded work during part of the past summer.

STEEL HULL

The hull is 80x25x5 ft., built of steel throughout. The frames run crosswise and consist of channels top and bottom with angle verticals. Two fore-and-aft trusses provide the necessary longitudinal stiffness, these trusses being connected directly to both the top and bottom members of the cross-frames. The bow and stern gantries are also of steel.

FIG. 1. ELECTRICALLY OPERATED DREDGE ON CUYAHOGA RIVER STRAIGHTENING, WITH ORIGINAL CUTTER

MACHINERY

All machinery is driven by 3-phase 60-cycle 2200-volt General Electric Co. motors, the variable-speed motors being of slip-ring type, equipped with 14-point reversing controllers. The current is brought on board by a submarine cable connected to transmission lines running parallel to the cut.

The main dredging pump has 16-in. suction and discharge openings and is direct-connected by means of a flexible insulating coupling to a 300-hp. motor mounted on the same bedplate. The pump is designed to deliver 225 cu.yd. solid material per hour through 1000 ft. of discharge pipe to 12 ft. above water, when running at 345 r.p.m. Owing to the great variation in the length of the discharge pipe required at various times, the motor was provided with controlling apparatus for running continuously with 25% speed reduction.

To sluice material into the cut ahead of the dredge, a 2-in. hydraulic giant is mounted on the bow of the dredge in such a way that it can be swung independently of the dredge and have a large radius of action. Water for this giant is provided by a 6-in. 3-stage Worthington turbine pump, delivering 1300 gal. per min. at a he of 360 ft., direct-connected to a 200-hp. motor operati at 1140 r.p.m.

A five-drum winch driven by a 20-hp. motor is u for handling the swinging-lines, raising the cutter l der and operating the spuds. The motor, fitted wi solenoid brake, is direct-geared through intermedia shafts to the drum shafts. Each drum is provided wi internal expanding friction and outside brake-band.

SUCTION LADDER AND CUTTER

The cutter drive and suction pipe are mounted on a structural-steel ladder suspended at the bow of the dredge on steel trunnions, through one of which the suction pipe passes; this joint is sealed to exclude air by means of water forced into the packing. The cutter is driven through reduction gearing by a 75-hp. motor running normally at 575 r.p.m. but able to operate continuously at half speed.

The cutter originally furnished (see Fig. 1) was of the heavy type with removable manganese-steel blades, but after the dredge had been in operation the subsoil

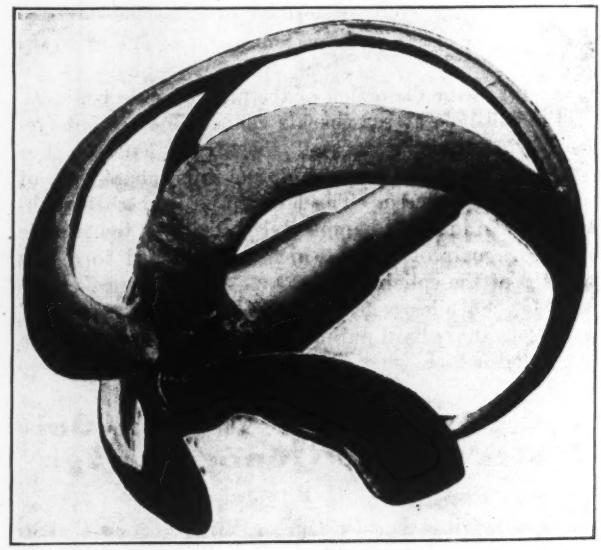


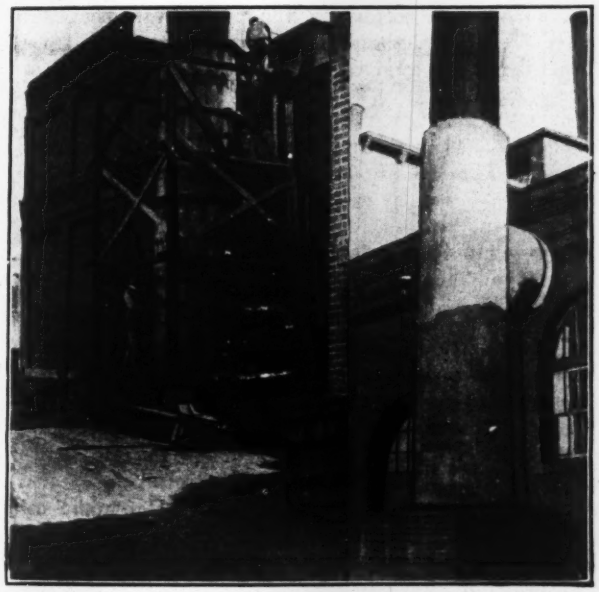
FIG. 2. FINAL FORM OF CUTTER FOR CUYAHOGA RIVER GUMBO

was found to consist of sticky blue clay or gumbo which clogged the cutter and refused to enter the suction pipe. A cutter was then installed having four knife-like blades, curved in the shape of an auger (as in Fig. 2, but shorter), and was found to do excellent work. This cutter was later replaced with one of the same pattern but of greater length (Fig. 2), as it was found difficult to properly gage the forward steps of the dredge with any great accuracy, the tendency being to overstep and leave uncut material behind the cutter.

A great deal of trouble and delay has been caused by logs embedded in the soil, which must be pulled out before the dredge can proceed. Some of these logs are 24 in. in diameter and 35 ft. long, and as high as six have been in the cut at one time.

Repairing a Badly Corroded Iron Chimney during Operation

A 90-ft. iron chimney at the Borough Electric Works, Wallingford, Conn., recently exhibited signs of corrosion at a point about 6 ft. above the base. Here for a height of 6 to 8 in. the metal, which originally had been $\frac{5}{16}$ in. thick, showed barely $\frac{1}{32}$ in. Circumstances made it imperative to continue the chimney in operation. It was decided that the most convenient way to strengthen the weakened portion was to jacket it with reinforced concrete. The illustration herewith shows the jacket under construction and completed.



FORMS IN PLACE (LEFT), AND SHELL COMPLETED (RIGHT)

Reinforcing rods of $\frac{5}{8}$ in. diameter were placed both horizontally and vertically, 8 in. c. to c. and 3 in. from the outer surface of the chimney; intersections were securely wired. A circular form, of diameter 1 ft. greater than that of the chimney, was then built up in sections and placed in position for pouring. The form was 25 ft. high. The concrete (1 : 2½ : 4½) was mixed very wet, owing to the high temperature to which it was subjected.

A few cracks have appeared in the concrete, but none serious.

A. L. Pierce, Superintendent and Electrical Engineer for the Borough of Wallingford, conceived this method of repair. The Sperry Engineering Co., New Haven, Conn., did the work in about four days. Mr. Pierce recently read a paper on the job, before the American Association for Advancement of Science.

A Concrete Floor Hardening System

A system which is being used extensively to give a hard and dustless surface to concrete floors consists essentially of a special method by which surplus water is quickly removed from the upper portion of the placed concrete, so that the surface may be finished before the cement has commenced to harden.

The usual methods of construction are employed until the wearing surface has been struck off level. Then burlap is laid upon the concrete. Cement and sand (or other fine aggregate), dry mixed in the proportions being used for the wearing course of the floor, are shoveled on top of the burlap and spread to a uniform thickness of about ½-in. Capillary attraction at once acts to draw surplus water from the concrete through the burlap and into the dry mixture. Only a very few minutes are required to carry this action to the point required. Then the burlap is lifted and the material upon it dumped into the mixing box where additional water is added, so the material can be used for the wearing course of other sections of the floor. In this way no material is wasted.

As soon as the burlap has been removed from the surface of the floor, the finishers proceed with their work. Under average conditions a floor finished by this method will support finisher's weight within 15 min. after the wearing surface has been placed, struck off level, and relieved of the surplus water.

It is believed that the dusting of concrete floors is caused by the breaking up of partially formed cement crystals in finishing concrete floors after the cement has commenced to harden. The above method was devised to overcome this difficulty and to make it possible to finish floors almost immediately after placing the wearing surface. It is the invention of P. M. Bruner, of the Granitoid Co., St. Louis, Mo., and has been employed on about a million square feet of floor in buildings in St. Louis, Kansas City and Chicago.

Putting Down Deep Foundation Piers in Long Island City

Carrying a five-story concrete factory building on concrete piers of depths varying from 3 to 28 ft. below cellar floor was found to be cheaper than any other method of footing at a job now under way in Long Island City.

The building is 180x88 ft. in plan and is carried on 52 piers averaging 3 ft. 6 in. square each. The first floor is 3 ft. above curb, the cellar floor 9 ft. 2 in. below curb, the pier caps 11 ft. below curb. The site, a pocket in the rock sub-base, was a swamp covered with from 2 to 2½ ft. of fill. The roads in the vicinity are constructed on fill, so that but little excavation was required for the cel-

lar. The principal excavation was for the piers, and this was done by hand.

Excavating for Piers

The pier centers were found by stretching galvanized wire across the lot in both directions and suspending plumb-bobs from the intersections. A board frame was then placed around each proposed pier, allowing 2 ft. extra width on each side. After digging down about 6 ft., the hole was sheetpiled with 2x8's, to which 6x6-in. stringers were fastened on the inside every 5 ft. downward as the work proceeded.

Disposal of Groundwater—Groundwater drained into the excavation from an area of about four blocks, making it necessary to pump out all excavations where work was in progress. The pumps were kept operating until the forms for the concrete piers were set, as the sheetpiling was not sufficiently tight to exclude the water. In digging the deepest hole, 28 ft. below the cellar-floor level, the pump was started at 3 a.m., and the men could not work in the hole before 10 a.m.

About five pier excavations were under way simultaneously, and there were five gasoline pumps on the job and four hand pumps for shallow holes. All pumps were

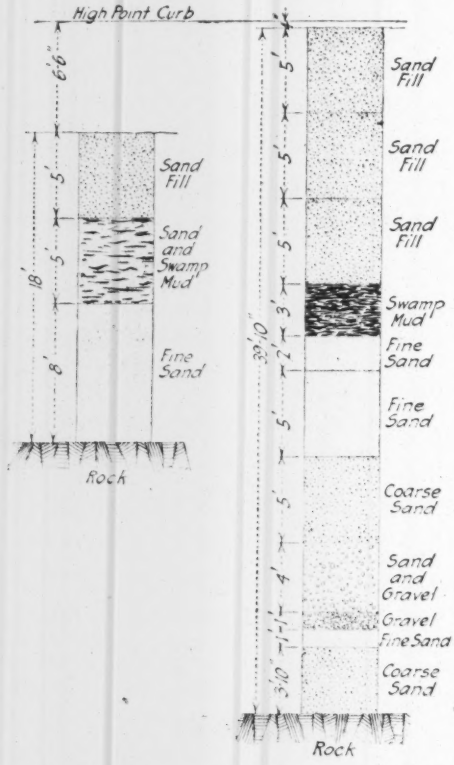


Fig. 1. Soil Encountered in Excavating, Shown By Distributed Test Borings

of the diaphragm type; one with a 4-in. discharge, the others with 3-in. A machinist was employed to keep the pumps in good condition.

Number of Men—For the first 7 ft., three men worked in the hole, while two on top removed the spoil. A platform was then built, and two men with shovels threw the spoil on the platform, from which a third man removed it. This continued to a depth of 14 ft., where a second platform was constructed in the hole, and one man worked below, passing the spoil to the lower platform, from which it was passed to the upper platform by a second man and removed thence by a third. A man outside the hole disposed of the muck as it was passed out. At the 20-ft. sublevel, a third platform was built, and the same method of excavating employed, requiring at this point a total of five men to the hole.

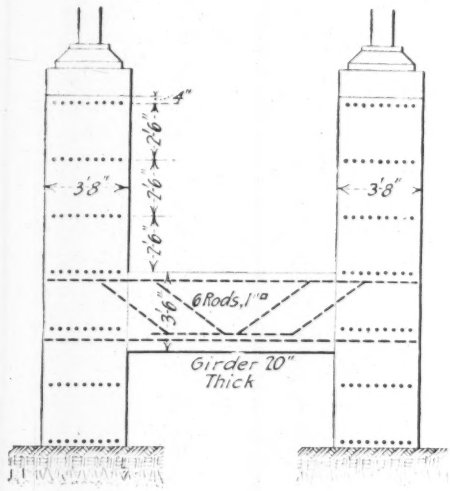


Fig. 2. Exterior Piers and Wall Girders

At the lowest level, the work was more restricted, of course, because the sheetpiling was bound to slope inward a little, so that the hole was a scant 7x7 ft.

The man at the bottom, or "the man with the boots," received $2 per day, as did also the man on the lowest platform; the others received $1.75 per day.

After Excavating

The rock, when reached, was blasted flat and a pocket formed. This operation was performed by two men receiving $2.25 per day each. The forms were panels built of 6x1¼-in. boards. Where two vertical panels joined they were braced with 4x4's wired at the corners.

The Concrete Piers

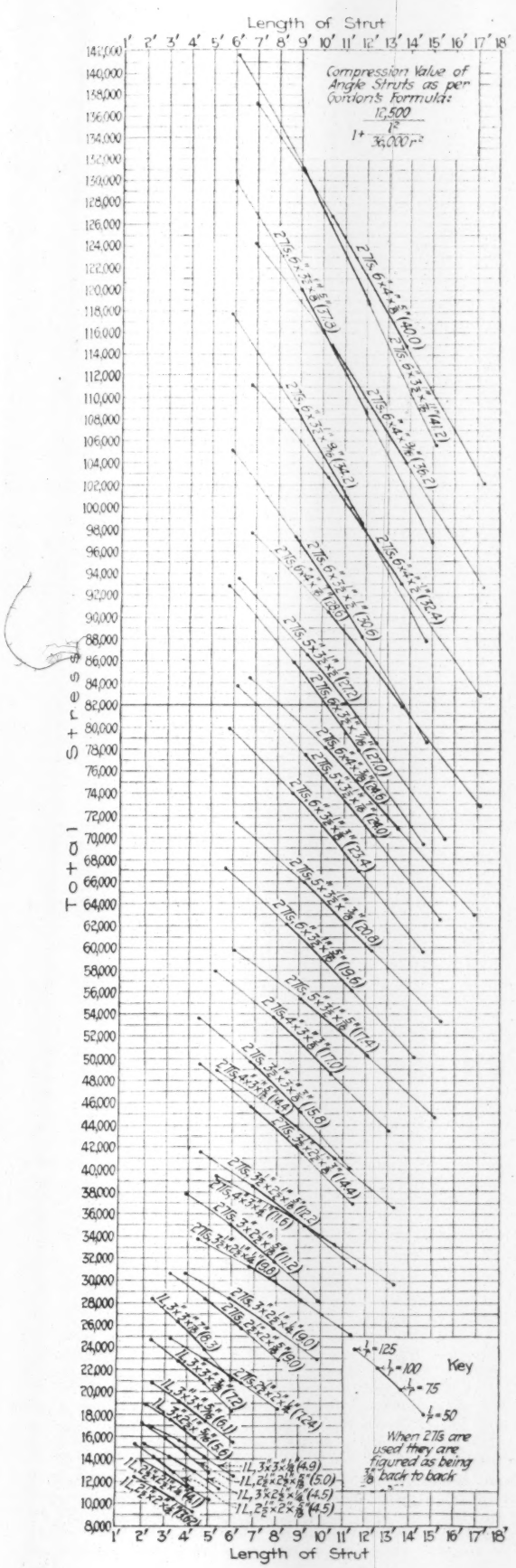
The pier reinforcing was entirely horizontal, to resist the tendency to diagonal shear. In the typical 3½x3½-ft. piers, sets of eight ½-in. twisted rods 4 in. apart were placed horizontally every 2 ft.

By the present building code of New York, a loading of 230 lb. per sq.in. is allowed on plain concrete in piers or footings. With the horizontal reinforcement mentioned, 500 lb. per sq.in. was allowed. Had plain concrete footings been used, they would have had to be 5x6 ft. in plan instead of 3x6 ft. for a load of 435 tons; and 4x5½ ft., for the interior piers, instead of 3½ ft. square, for a loading of 328 tons.

Two concrete mixers were installed at street level. The concrete was chuted down into the pit, and conveyed by wheelbarrows to the piers. Each pier was allowed to set three days, when the forms were pulled and the space backfilled. Only the upper row of sheetpiling was pulled.

The curtain walls were carried on reinforced-concrete girders between piers at the cellar-floor level (see Fig. 2).

The work was started on Sept. 3. On Nov. 18, 47 piers were finished and five were under way. The work is


Length of Strut
Compression Value of Angle Struts as per Gordon's Formula:
Key
When 2 ℄s are used they are figured as being ⅜" back to back
Total Stress
Length of Strut

being done by the Evergreen Construction Co., Long Island City, whose superintendent on the job, and likewise treasurer, is E. W. Fitzpatrick. Maurice Deutsch, of New York City, is consulting engineer.

Diagram for Capacity of Angle Struts

A useful variation from the numerous tables of angle struts is found in a diagram made by Simeon O. Johnston, Walkerville, Ont., and reproduced herewith. As printed here it is reduced several times from the blueprint original, but its scale still permits of making precise readings.

The diagram is based on a formula of the Gordon type. The author thinks that the value of the diagram is not greatly dependent on the particular formula used, because all current formulas nearly coincide in the range between length-ratios 75 and 100.

Economical Paving with 3-In. Brick

In several small cities where a substantial paving is desired, but where brick paving of the ordinary construction is considered too expensive, successful results have been obtained with vertical-fiber brick having a thickness of 2½ or 3 in., instead of the standard 4-in. thickness. A case of this kind was described in a paper by G. E. Johnson, City Engineer of Falls City, Neb., presented at the annual meeting of the Illinois Society of Engineers and Surveyors at Springfield, Ill., Jan. 27-29.

The surface was first graded and rolled, all trenches being rolled to a surface at least 2 in. below subgrade in order to provide an extra thickness of concrete. The concrete base was normally 4 in. thick, and on it was a 1½-in. sand cushion. The bricks were 3x4x8¼ in., with square edges, and were laid flat on the sand. This paving was rolled and after the bricks had been inspected and culled the joints were filled with an asphalt filler poured on with common coal buckets and spread by means of squeegees. A thin layer of coarse sand was spread on the pavement immediately after the filler was applied. The cost is given in the accompanying table.

The selection of a filler for this paving was a difficult proposition. One with a melting point of 250° F. was recommended, but proved to be too hard. The filler adopted has a melting point of 170° and was poured at a temperature of 400° F. It is found to stand various conditions of traffic and temperature without getting loose or becoming soft enough to stick or track. The hard filler first recommended was used on a 30-ft. street carrying from 300 to 500 tons of freight per day. Although a portion of this filler has chipped off the surface of the pavement remains smooth and quiet.

TABLE OF COST PER SQ.YD. FOR SHALLOW BRICK PAVING

Excavating	$0.03
Surfacing and rolling	0.015
Cement ($1.63 per bbl.)	0.185
Stone ($2.25 per yd.)	0.232
Sand for base ($1.10 per yd.)	0.06
Mixing and placing	0.05
Sand cushion ($1.10)	0.046
Fiber brick, f.o.b. cars at city	0.70
Unloading and hauling brick	0.05
Preparing cushion and setting brick	0.04
Rolling brick	0.006
Filler	0.12
Applying filler	0.03
Incidental expenses	0.017
Total, per sq.yd.	$1.581
Contract price	$1.73

NOTES

Stone Screenings as Fine Aggregate for concrete are not favored by the New York State Highway Department. As a result of some field tests of concrete in which all the fine aggregate was stone screenings, it was decided that the only allowable use of such screenings should be in conjunction with a much larger percentage of sand. It was found that a proper proportion of screenings from a hard stone will improve a fine-grained sand but the high cost of proper inspection and mixing makes the use of such a mixture prohibitive, except where a good sand is very expensive. Laboratory tests of screenings as aggregate showed up much better than the above-noted field tests.

Vicissitudes of Surveying Instruments—Almost every engineer can recall some incident in his surveying experience where his instrument survived an accident which threatened its disintegration; and the question often arises just how badly damaged an instrument can be and still be repaired and entitled to be called the same instrument. Such a case is Fig. 1, which shows what was once a Buff transit. The parts shown are what survived the Salem, Mass., fire of last June. This transit was returned to the manufacturer "for repairs." Fig. 2 is a dumpy level hit headon by a freight train traveling so rapidly that the levelman had to make his exit from the track without stopping to gather up his instrument. Strange as it may seem, the level vial is still intact, and the damage to the instrument as a whole is by no means beyond repair. These instances prove that with occasional "necessary repairs" a beloved "gun" may be made to outlive its owner.—Photographs from Louis Buff, Boston, Mass.

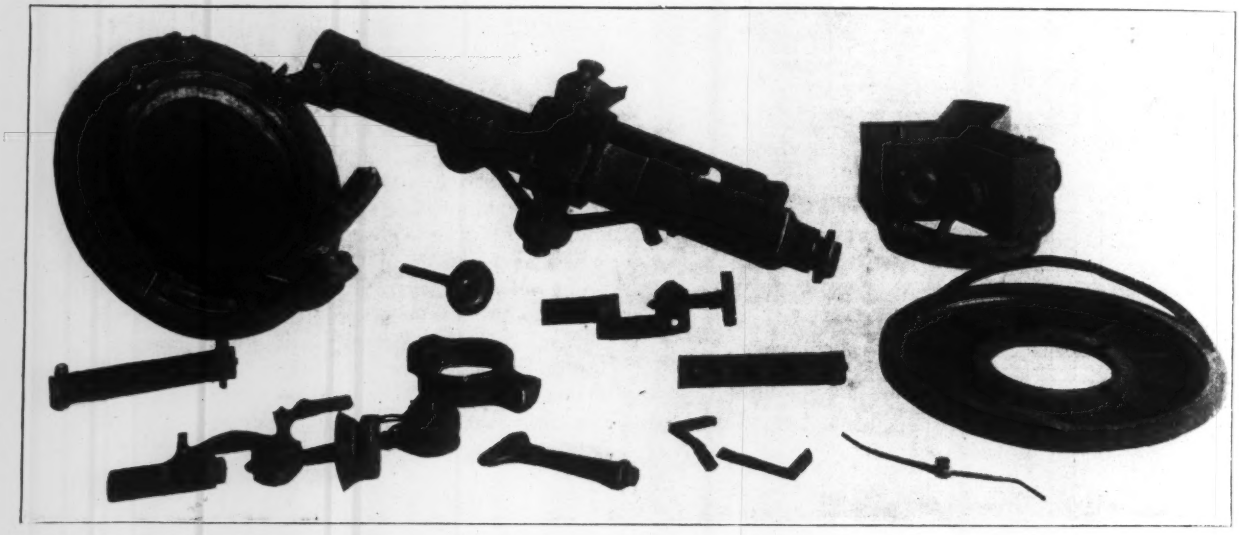


FIG. 1. REMAINS OF AN ENGINEER'S TRANSIT WHICH WENT THROUGH THE SALEM FIRE

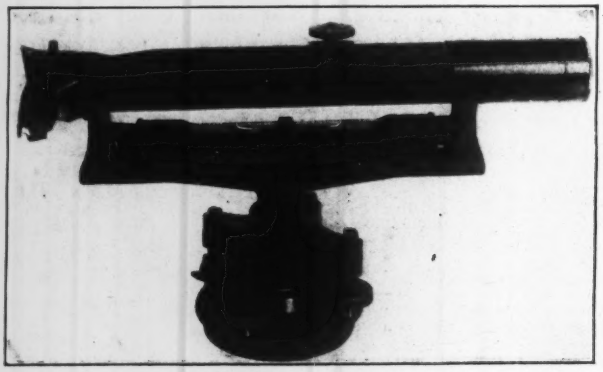
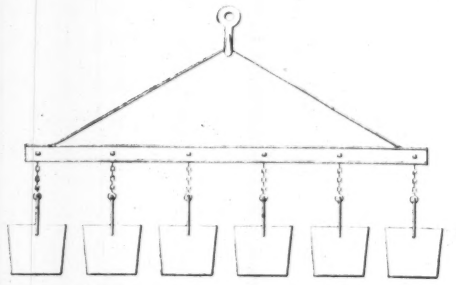


FIG. 2. A DUMPY LEVEL AFTER BEING HIT HEADON BY A FREIGHT TRAIN

A Unique Device for Handling Concrete Buckets—In building the Keokuk dam a considerable quantity of concrete [illegible] to be placed in the turbine pits during the winter of 191[illegible]. The foundation rings of the 10,000-hp. turbines were [illegible] concreted in. Concrete was brought from the mixing [illegible] on the opposite side of the river, carried across the mile-[illegible] dam by the contractor's railway, and handed along the 900-ft length of the generator room by two slow-geared 150-ton cranes which serve the machinery in this room. Handling the 1¼-yd. buckets singly by means of these cranes was very slow work, which proved particularly bad on account of the freezing weather. It was necessary to find a more rapid method of handling the buckets from the train along the generator room.



SPREADER FOR CARRYING SIX 1¼-YD. CONCRETE BUCKETS, KEOKUK DAM

The foreman in charge solved the problem thus: He took a long steel beam built up of two channels spaced by latticing, and inserted bolts in the web to attach six heavy chains and hooks. A cable sling was fastened to the ends of the beam for suspending it from the crane hook. The sketch above shows the rig in outline. This device formed a long yoke or spreader, capable of picking up and carrying six of the buckets at a time. The buckets came twelve on a train. When the train came in, six of them could be picked up at once and moved along by the crane to the dumping platform. Here the six full buckets were set down and a string of empties picked up and carried back to the train. While these were going back the second crane served the dumping platform, one bucket at a time, and from the platform the concrete was shoveled down by hand into a vertical shaft, falling 35 to 40 ft. to the turbine pit. When the second set of six full buckets arrived the first set was empty and ready to be returned.—H. B. McDermid, Cambridge, Mass.

Home-Made Steel-Protected Concrete Curbs are made in Meadville, Penn., as follows: Steel-protected curb used on all radii is 1:2:4 concrete, 6x24-in., protected by a strip of tire steel, ½x4-in., bent to the proper curvature and held in position by countersunk bolts, with the nuts left in place in the concrete.

Editorials

A Lesson Partly Learned

Experience is the great teacher. Previous to last October, the City of Youngstown, Ohio, had absolutely no regulations governing the design and construction of reinforced-concrete buildings. On the 26th of that month, a reinforced-concrete building under construction there fell down and killed three men. Under date of Jan. 11, 1915, the City Council added a section to the building code embodying the most approved views of the supervision of reinforced-concrete work, including the radical provision for a special concrete inspector, employed by the contractor, but approved by and reporting to the Inspector of Buildings.

Such belated reform is exceedingly human. Possibly in this case the city officials will make the superhuman effort to enforce the new regulation and not fall into the common habit of thinking that rules and codes are self-operative.

Developments in Concrete Testing

The testing of concrete is today a very different matter than it was a few years ago. In the early days of concrete construction, such testing was primarily to establish unit-strength values which could be used, with proper safety factors, in design. After a sufficient number of such tests had been recorded, attention turned toward the investigation of different conditions of materials, mixing, and exposure, the results from which were to be applied to the problems of construction. The purpose of all such tests was to supply general information.

Within the past few years, however, a change in testing has taken place. Today, by far the greater part of concrete investigations are directed toward two ends: the investigation of the availability of aggregates for some specific concrete work; and the testing for strength of concrete taken from actual work as a guide to the integrity of the structure.

In this issue we publish two articles covering, one very completely and the other in brief, two separate examples of this later development. The article by Mr. Goodwin on the concrete testing laboratory now being conducted in connection with the enormous subway contracts under way in New York City is the most complete outline yet published of the details of work in a laboratory conducted solely with a view to work about to be undertaken or under construction, and not with any specific consideration to the general problem of concrete construction. While the extreme detail demanded by such an extensive work will hardly apply to minor projects, still the very elaboration of the detail makes it useful to the engineer who contemplates—as all engineers on such work should contemplate—the study of the concrete which is going into the structure.

The second article, a brief extract from a longer society paper, by H. S. Mattimore, outlines some extraordinary tests conducted by the New York State Highway Department in connection with the concrete work now going on under that department's direction. The figures as to comparative value of different aggregates for concrete are unique and undoubtedly valuable, but the most striking feature of the report is the extraordinary high values for certain field-made concrete tested in compression.

Some months ago we presented some figures of field-made concrete which showed that, in one particular city at least, highway concrete hardly reached 1000-lb. per sq.in. compression value at the end of a month. Some of our friends engaged in the concrete industry were somewhat concerned at this article and wrote us saying that the impression given was quite erroneous, in that the tests were not representative and that most field tests have shown so high unit stresses that the usual ultimate unit value of 2000 lb. per sq.in. for 28-day, 1:2:4 concrete is conservative. While such tests as have come to our attention seem to show that 1500 lb. per sq.in. would be more nearly the correct figure in this case, the claims of these correspondents would certainly be bolstered up by Mr. Mattimore's tests, which show unprecedented unit strength. In his "Concrete-Steel Construction," Mörsch recounts some tests made in connection with a bridge over the Danube at Munderkingen at which some long-time compressive strength tests of concrete showed for 1:2½:5 mix and 8-in. cubes, a compressive strength of 2873 lb. at 7 days, 3613 lb. at 28 days, and 7396 lb. at 2 yr. and 8 mo. So far as we know, this is by far the highest strength value ever recorded for concrete work, until these recent tests by Mr. Mattimore, where 1:1½:3 concrete 6-in. cubes show compressive strength for approximate 30-day periods varying from 3980 to 5555 lb. per sq.in. (equivalent to an average 28-day value of 4350 lb.).

For the best kind of concrete of such proportions the Joint Committee cites 2800 lb. per sq.in. as a maximum compressive value. The undoubted authenticity of the New York State tests warrants their inclusion in the history of concrete testing, but they are nevertheless decidedly freaky and should be so considered unless some further reasons are advanced for the increased strength, or until similar tests show comparative results. At any rate, it seem too bad to waste such strong concrete on a highway.

Wind Pressure and Wind Stresses

What do we know about the action of wind on buildings and bridges? Very little indeed! It is useful to be reminded occasionally of our ignorance in this subject, as was done in the article of R. Fleming, printed in last week's issue of ENGINEERING NEWS. The truth is that much money is spent in providing strength to resist wind action, and this expenditure is based on a mixture of empirical rule with certain alleged theoretical and experimental knowledge that is little more than bold guesswork.

Next in value to saving money is knowing for what we spend our money—or, in the present instance, for what we *think* we spend it. As Mr. Fleming set forth with embarrassing clearness, we know very little about wind force and its variation with roof slope, height and shape, but we pretend we know much. He politely convicted us of rank foolishness in accepting and using the vainly mathematical formula of Hutton for pressure on a sloping surface, a formula whose sole basis is a set of experiments with a toy-size apparatus subjected to mild breezes—12 miles per hour. Similar pretense of knowledge erected on the flimsiest kind of foundation appears elsewhere in the field of wind-stress analysis.

In the present issue is printed a further article of the series. While dealing with stresses in roof-trusses and mill-buildings instead of with wind pressure *per se*, it repeats the story of our ignorance as to wind actions. It reviews the tests on roofs which show complex distribution of upward and downward pressures, of a kind to render valueless all refined assumptions as to maximum wind-pressure and the like. But it shows also—and this is a highly practical demonstration—that simple empirical assumptions, used in a method of design nearly identical with what has been common practice for years past, will give safe results.

Heavy Motor Traffic on Public Roads

Every engineer interested in highway construction agrees that the most difficult problem in highway work at the present day is to provide for the maintenance of a road after money has been spent on its improvement. What railway company would permit its traffic manager and superintendent of motive power to purchase and operate locomotives and cars without regard to the strength of bridges on the road or the weight of the rails in its track? This, however, is exactly the policy which is being pursued, almost without exception, by the various States in connection with their improved roads. From $5000 to $10,000 or even $12,000 a mile is spent on grading and surfacing and improving a road, and then it is opened to traffic and anyone is at liberty to run over it vehicles of any weight he chooses and at any speed.

The situation as it affects bridge design was ably reviewed recently in our columns by Willis Whited, of the Pennsylvania Highway Department. Its relation to the destruction of the road surface is discussed in the annual report of Commissioner Carlisle, of New York, sent to the legislature Jan. 18. New York has spent nearly or quite as much on state road construction as all the rest of the states combined and these roads are rapidly being destroyed by improper traffic. The Commissioner of Highways of New York has, it is true, adopted rules and regulations as to the vehicles which may be run over state or county highways; but it is one thing to make such regulations and another to enforce them. The Highway Commissioner has no authority to punish violations of these rules. The bringing of suits against violators rests with town and county officials, who take little interest in the matter and are not at all inclined to hire attorneys to enforce the law. The Commissioner, therefore, very properly urges that traffic regulations for the improved roads of the state should be enacted by the legislature and that authority should then be given to the State Highway Department to see that the law is enforced.

Similar difficulties with damage to improved roads are reported in other states. Operators of traction engines used for threshing and hauling often run them over a road when their wheels are armed with spikes and lugs which do serious injury to its surface. The most serious damage to improved roads, however, comes from the rapidly increasing use of the motor truck and the motor omnibus. Already, Commissioner Carlisle declares, the motor omnibuses which are now running on practically all the state roads of New York, are doing more damage to these roads than any other vehicles. These motor buses carry heavy loads and are run at speeds which very rapidly tear the roads to pieces, especially in the spring and fall when the roads are soft. Similar conditions exist in New Jersey and in other Eastern states which have spent large amounts of money in the improvement of roads. These vehicles are competing with steam and electric railways and are practically using the highways which the state has built as roadbed and track and paying nothing for their use. It is beyond all question that such vehicles should be subject to reasonable regulations as to the weight, speed and character of tire used, and there is much to be said in favor of the proposition that those operating them should pay a reasonable fee for the use of the roads, sufficient to make good the damage which they cause. The only alternative, unless the roads are to be rapidly destroyed, is that the already overburdened taxpayer must furnish the money to make good the damage.

Of course the owners of motor omnibuses claim that the motor trucks, which are also very rapidly increasing in number, are as much responsible as the omnibuses for the wear and tear of roads, and declare that if one class of vehicle is taxed for the use of the roads, all others should be taxed. Hitherto every proposition for special taxation—and even the proposition to limit the weights and speeds of motor vehicles—has been strenuously opposed by the builders and users of these vehicles. Such opposition, however, appears to be extremely shortsighted. The good roads movement will receive a serious setback if the destruction of the roads already built is allowed to continue, or if the taxpayers are asked to bear unaided the heavy burden for maintenance which is at present necessary. There is no doubt that the motor truck and the motor omnibus are here and here to stay; but they must be so used as to do the least injury to the roads over which they run and they should bear their fair share of the cost of maintaining these roads.

The Engineering Foundation

A gift of $200,000 "for the advancement of the profession of engineering and the good of mankind," is surely a notable event in the annals of engineering. Those who have the privilege of knowing personally the generous donor of this gift, Ambrose Swasey, of Cleveland, will appreciate that both the gift and the manner in which the gift was made are characteristic of the man.

The first comment to be made regarding this benefaction is that it ought to have no small influence toward furthering coöperation and harmony in the engineering profession. It has been said, and said by engineers of prom-

instance, that engineering is not a profession at all, but a business. It used to be said, too, that the civil engineers had the chief claim to rank as members of a profession, and that mechanical engineers were either skilled workmen or business men engaged in manufacturing. That old prejudice has entirely disappeared, we hope, but it is noteworthy that this generous gift, made for the benefit of the whole engineering profession, comes from a mechanical engineer.

It is a matter of history that when, nearly a dozen years ago, Andrew Carnegie made his generous offer to give a million dollars (afterward increased to a million and a half) to erect a great building in New York City as a home for the four great national engineering societies, it was the hope of those who had planned this movement toward coöperation among engineers that all four of the great national societies would accept this generous offer and take advantage of the splendid facilities thus made available.

It is a matter of history, also, that the American Society of Civil Engineers, in response to the leadership of some excellent but very conservative old men, who have long since passed away, decided not to accept the offer; and that as a result the Civil Engineers' Society, justly or unjustly, has been regarded to a certain extent as holding a sort of I-am-holier-than-thou attitude toward the other national societies. So far as this attitude and belief have existed, it has certainly been unfortunate for the engineering profession.

It is, therefore, a matter of exceeding interest that when Mr. Swasey's generous offer was made to the Trustees who represent the three national societies which own the Engineering Societies' Building, it was decided that the gift should be administered by trustees, selected not alone from these three national societies but from the American Society of Civil Engineers as well. The Board of Direction of the American Society of Civil Engineers formally accepted this offer.

It is to be hoped, as was expressed by one of the speakers at the meeting celebrating the inauguration of the Engineering Foundation, that the coöperation of all the four Societies in the administration of this gift, made for the benefit of the profession as a whole, may mark the final disappearance of any barriers of prejudice between the American Society of Civil Engineers and the other great national societies.

The question which at once arises in connection with this generous gift of Mr. Swasey is what use is to be made of the money, or rather of the income from the money, since under the terms of the gift the principal is to be kept intact as a permanent fund. The by-laws of the United Engineering Society leave this matter to the discretion of the trustees of the Engineering Foundation as follows: "The board may use its funds in any manner it deems proper for the furtherance of research in science and engineering, or for the advancement in any other manner of the profession of engineering and the good of mankind."

It seems to be generally assumed that what the engineering profession needs most is research. We make bold to say that the engineering profession has far more research at its disposal today than it can use. Dr. Henry S. Pritchett, in his address at the inauguration exercises, paid his respects to "research," as it is generally carried on in the universities and technical schools of the United States, and intimated that a very large proportion of the work now being done under the name of research in the United States represents time and money wasted.

We do not mean to say, of course, that the great field of knowledge has been wholly explored, or even that limited part of it which directly affects the work of the practicing engineer. It is a fact, however, that the American engineer or manufacturer who has a practical problem which needs thorough investigation by a physical or chemical laboratory has at his disposal, besides a number of commercial organizations thoroughly equipped to carry on such work, a large number of laboratories in connection with the universities and colleges, and in addition the great laboratories of the Bureau of Standards at Washington, to say nothing of a number of other bureaus of the Federal government which engage more or less in scientific research. Under these circumstances, to devote the $8000 or $10,000 a year, which will be the present income of the Engineering Foundation, to additional work in scientific research would be merely adding another drop to a bucket already well filled.

But, it may well be asked, what then can be done with this money? How can it be expended to yield some real benefit to the profession and to the public? We shall not attempt within the limits of space here available to give a complete answer to this question; but we venture to offer at least a suggestion, and our suggestion is that what most needs investigation today is not the materials used by the engineer or the methods he employs, but the conditions under which engineering work is being done. Engineers everywhere are dissatisfied with the status of their profession. They desire to see their work appreciated as it should be by the public. They are aware that the public which employs engineers often does not know what to expect of an engineer; and is on the one hand unappreciative of good work, and on the other hand ready to accept very poor work because it has no means of knowing what good work is.

Let us give a concrete illustration: A prominent engineering society has recently undertaken to prepare a brief pamphlet, stating in simple, plain terms just what are the duties which an independent practicing engineer is supposed to perform for his clients, under what conditions he is supposed to work and what are the advantages which the client will gain by employing him. The pamphlet, when it is in satisfactory shape, is to be given wide publicity and to be made available to engineers for circulating among their clients or possible clients.

In other words, these engineers have realized that for the betterment of the engineering profession it is necessary that the profession should take up the work of instructing the public—its clients—and that this is a work which can be far better done by engineers acting together than by any engineer individually.

Several papers have been read recently before national engineering societies dealing with the need for advertising the engineering profession. The individual engineer cannot advertise to any extent; his business is too limited and his income too small to permit this in the first place, and in the second place the need is for advertising to the public what engineers are able to do and not what the especial individual engineer can offer.

Let us take another concrete example: In the mechanical engineering field, it is a very common complaint that

the manufacturers have to a large extent driven the independent designing and consulting engineer out of business. Something similar happened years ago in the field of bridge engineering. The design of an ordinary span highway bridge was once a matter for which an individual engineer would prepare drawings, specifications and complete designs. This work soon came to be done as a matter of ordinary routine in a bridge building company's office at far less cost than any independent engineer could afford to do it.

The pendulum has swung too far, however. The tendency nowadays is for buyers of machinery and equipment to rely altogether too much upon the manufacturer for engineering design. Manufacturers themselves, in many cases, would prefer not to undertake this work. It is really done wastefully, since the purchasers generally insist on a large amount of such designing and planning being done by each of the bidders on a project, so that each manufacturer has to undergo a large expense for which one only—the one receiving the contract—can receive a return.

The client would be better served if he could have the services of a first-class engineer to give to the entire problem the study it deserves and adopt the solution which will be best for the client's interests and not that which suits the commercial convenience of one particular manufacturing concern. The manufacturer would have a better chance to do an honest and a profitable business than he has under the present situation when he is compelled to compete to furnish the customer with free engineering advice; and the engineer would have an opportunity to make an honest living in the practice of his profession. This is the situation, a situation so serious that a prominent consulting engineer in New York City, of wide reputation and large experience, not long ago expressed the opinion that if present tendencies were to continue unchecked, the independent consulting engineer in his particular field would soon become extinct.

There is no doubt that the individual engineer is practically helpless to remedy this condition, except in a limited way, dealing with a few individual clients whose confidence he has won. Manufacturers, also, seem unable to change the situation which has developed as a result of their competition, and the public, of course, doesn't know.

Is it not possible that an organization with the proper financial backing might deal with a situation of this sort, and find some way to educate the public to the value of the engineer's services in this field of work?

We submit these random suggestions as to the class of work that the Engineering Foundation might usefully undertake, and hope those in charge of it can plan an intelligent, workable scheme that will result in real benefit to the engineering profession and the public. If such a plan can be developed, there is no doubt that ample funds can be readily obtained, if needed, to add to Mr. Swasey's generous gift.

Letters to the Editor

Flagpole Bursts; Condensation of Moisture in Idle Stack

Sir—The following item from the North Platte *Semi-weekly Tribune*, describes the bursting of a steel flagpole at the Weather Bureau:

With a crash that was heard for some distance around, the flagpole at the weather bureau office yesterday morning about 6:30 o'clock, burst and broke off about 10 ft. above the ground, falling across the sidewalk. No serious damage was done except to the flagpole which will have to be replaced.

The bursting of the pole was due to it being full of water for a distance of 25 ft. above the ground. The water froze and burst the heavy iron casing. It burst with a crash and a piece of the pipe was blown out some distance away from the pole. The flagpole is made of heavy gas pipe. It is 4 in. in diameter at the base and tapers to a ¾-in. hole at the top. How it could fill with water for a distance of that height remains a mystery.

Weather Observer A. W. Shilling is at a loss to explain the phenomenon. He has made calculations and found that all the rainfall that could have fallen through the small hole at the top since the pole has been standing would fill the base up only about 2 ft., and that is not allowing for what would seep away through the bottom. It is possible that the condensation of vapor on the inside would add a small amount, but what could fill it up so high remains a mystery unless it could be that the pipe is connected in some manner with an artesian well.

It is possible that moisture condensing on the outside of the pole ran down inside at the joints made by the smaller pipes with the larger if these joints were not close.

A concrete smokestack 150 ft. high was recently finished for the North Platte Electric Co., and before the connection was made with the plant a cleanout door in the bottom was left open. On mornings when the air was humid, the difference in temperature between bottom and top of stack was such that the rising column of air had its moisture condensed at the top of the stack and this fell in a considerable shower, lasting for an hour.

CHARLES S. STEWART.

North Platte, Neb., Jan. 15, 1915.

Experiments to Determine the Cause of Dusting of Concrete Floors

Sir—Noticing an article in regard to the dusting of floors in a recent issue of ENGINEERING NEWS, I send you herewith a report of some experiments we have carried on ourselves, which may be interesting to your readers.

W. H. HARDING, President,
Whitehall Cement Mfg. Co.

Land Title Building,
Philadelphia, Penn.

Whitehall Cement Manufacturing Co.

Referring to the experiments which we have been conducting during the past three weeks, relative to the behavior of various mortars in proportions of 1 part Whitehall portland cement to 1 of sand, up to 6 parts of sand to 1 of cement, we beg to submit for your consideration the following results:

1 : 1 mixture (moist) wherein only enough water was added to hold the mortar in place and not bring any moisture to the surface on tamping, and which, in ordinary practice, would be considered dry, was, on hardening, found to be

no dusting. 1 : 2, 1 : 3, 1 : 4, 1 : 5 and 1 : 6 mixtures, when treated in a like manner, showed the same general result, there being absolutely no dusting of any of the test pieces at any time during the three weeks period.

Our next experiment was with plastic or normal mixtures —the first set of samples being troweled moderately on filling the forms; one hour later the surface of each was again troweled moderately. All mortars from 1 : 1 to 1 : 6 inclusive failed to show any signs of dusting.

The same plastic mixture was next troweled excessively when placed in the forms; this procedure brought considerable water to the surface and presumably much of the cement. In one hour's time the surface of each test piece was again lightly troweled to excess, with these results:

1 : 1 slight dusting.
1 : 2 dusts freely.
1 : 3 dusts freely.
1 : 4 dusts freely.
1 : 5 dusts } very lean mixtures, which accounts for same showing less dust.
1 : 6 dusts }

The third piece of work was with very wet mixtures. These samples were troweled lightly when put in the forms. Two hours later the surface of each test piece was again moderately troweled, which after hardening yielded the following:

1 : 1 dusts freely.
1 : 2 dusts freely.
1 : 3 dusts freely.
1 : 4 dusts freely.
1 : 5 dusts.
1 : 6 dusts.

The conclusions to be drawn from the foregoing experiments are that excessive troweling will invariably cause the surface of a floor to dust, regardless of the proportion of sand and cement used, when either plastic or wet mixtures are employed.

Furthermore, wet mixtures for this purpose are not to be recommended, on account of the water carrying too much of the cement to the surface and holding it there while the mass is still fluid.

Troweling the surface two hours after the same is finished is also to be avoided, as, in most cases, the initial set, in that length of time, is approached and thereby disturbed, which has a tendency to retard the normal set of the whole mass with resultant dusting.

HARRISON & SCHAFFER.

Easton, Penn.

Cost of Cement, Los Angeles Municipal Plant

Sir—In a report by the Board of Public Works of the City of Los Angeles to the Mayor and City Council on Dec. 10, 1914, occurs the following statement:

The efficiency director verbally explained that he estimated the cost of manufacturing cement at the Monolith plant at $1 per bbl., and 40c. per bbl. for freight from the plant to Los Angeles, making the $1.40 per bbl. From the beginning of operation at the Monolith mill, in March, 1909, to Mar. 31, 1913, there were 891,648 bbl. of cement manufactured, at a cost for operating expenses of $1,689,292.24, or an average cost of $1.894 per bbl. This includes periods when the mill was running on part time and also all repairs and replacements, and 10c. per bbl. for depreciation of plant. It does not include anything on account of the investment in the plant, which was about $850,000.

The above is in contradiction of the claims made by the chief engineer of the Los Angeles aqueduct, that this plant can manufacture cement for $1 per barrel at the mill. The Board of Public Works in its report admits that the only way such a unit cost has been, or can be obtained, is by selecting short periods from the accounts of the city. This would be manifestly unfair, as a period when little or nothing was actually expended for repairs and maintenance could be selected, so as to give an erroneous impression.

Further, in the statement made by the Board the depreciation on the plant is only given as 10c. per bbl. of the output, which would be less than $90,000 total depreciation. There is little doubt that in the six years during which the plant has been operated the depreciation thereon, including that on cement sacks, has not been less than three times $90,000. If this correction be made and annual interest on $850,000 investment be allowed, the cost of cement at the Monolith plant will reach approximately $2.50 per bbl., to which adding freight to Los Angeles of 40c. per bbl., will make the cost $2.90 per bbl., whereas the open market offers the best portland cement at wholesale for approximately $2 per bbl.

In order to make the above showing the total output of the plant has been made to include all the cement ever manufactured, whether usable or defective. A large amount of the 891,648 bbl. has been defective cement, has never been used, and should be charged to profit and loss. This would still further increase the cost of cement produced at the Monolith plant and used in connection with the Los Angeles aqueduct.

F. C. FINKLE.

Los Angeles, Calif., Dec. 12, 1914.

[This letter was submitted to the Los Angeles Department of Public Service which has replied as follows, through O. E. Clemens, Cost Accountant.—EDITOR.]

Sir—The average cost of $1.894 per bbl., noted in the recent report of the Board, includes all expenses of the mill operation, including preliminary try-out expenses, depreciation at 10c. per bbl., and all repairs and maintenance and other expenses, including all losses on cement sacks from March, 1909, to October, 1910. Beginning in October, 1910, the mill operating sack account was changed to conform with private mill accounts and each construction division of the aqueduct was charged with the sacks shipped to it, credit being given for return only of usable sacks, the loss on damaged and destroyed sacks being borne by the construction. All costs of actual repairs and waste at the mill was charged to and is included in the above operating cost.

The general statements made by the efficiency director regarding the cost at which cement can be manufactured at the mill, were misapplied by him in his statements to the Board of Public Works. The general statement was made that under private management this mill could undoubtedly produce cement at $1 per bbl. or less. This unquestionably can be done, if the restrictions of the 8-hr. working day and other drawbacks in connection with municipal operation of a mill such as ours were removed by placing same under private management.

When the mill was running full capacity, an operating cost of $1.10 to $1.15 per bbl. was obtained, which included all normal repairs and percentage, but no depreciation charge. These costs were maintained under the rigid city requirements of an 8-hr. day and with the rock quarry not so advantageously situated as the present one. The present quarry was brought to a good operating condition only a short time before the mill was closed down on account of the completion of the aqueduct.

Mr. Finkle's statement as to defective cement is highly exaggerated and not based on fact. The comparatively small amount of rejected cement from the Monolith mill, amounting to less than 2000 bbl., was handled in the same way as that purchased of private concerns was handled. The mill operating account was charged with all expense caused the construction division by shipment of defective cement, even to the expense of freighting, handling and waste sacks, which, be it noted, Mr. Finkle wants to charge for again, when they are already charged for in the figures he purports to use.

O. E. CLEMENS,

Los Angeles, Calif., Jan. 8, 1915.

Progress on the Greater Winnipeg Aqueduct*

The year 1914 closed with work well under way and up to schedule on the 97.2-mi. aqueduct of the Greater Winnipeg Water District. During the year permission to divert water from Shoal Lake, a part of the Lake of the Woods, was secured from the International Joint Commission, surveys previously begun were completed, the right-of-way was secured, the construction railway was all but completed, the aqueduct structures were designed, the contracts for the aqueduct proper were let, other work was accomplished, and provision was made for financing the project.

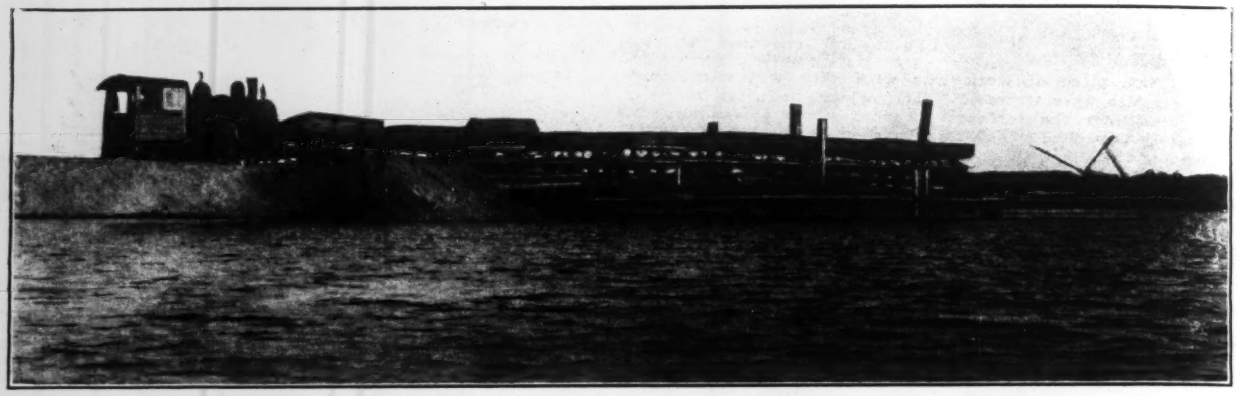


FIG. 1. BUILDING THE DIVERSION DIKE AT INDIAN BAY FOR GREATER WINNIPEG WATER-SUPPLY
(The scow was used to run the cars onto after dumping)

Briefly, the aqueduct will supply 85,000,000 Imp.-gal. (102,000,000 U. S. gal.) a day to the cities of Winnipeg and St. Boniface and to all or parts of five other towns surrounding Winnipeg, all at an estimated cost of $13,500,000. The work includes 104.2 mi. of railway track for construction purposes, a diversion dike and channel, an intake, 35 mi. of cut-and-cover aqueduct from the intake to the site of a future reservoir southeast of Transcona, 9.8 mi. of 60-in. steel pipe from the reservoir site to the Red River, a tunnel beneath the river, and 2.3 mi. of 48-in. cast-iron pipe from the river to the McPhillip's St. reservoir. Advantage is taken of a gross fall of 294 ft. between Shoal Lake and Winnipeg to bring the water in by gravity.

The project was approved by popular vote on Oct. 1, 1913. Thereupon, S. H. Reynolds, a civil engineer, was appointed chairman of commissioners; J. H. Ashdown, a commissioner (without compensation); James H. Fuertes, of New York City, consulting engineer, and W. G. Chace, chief engineer.

Five survey parties were speedily placed in the field for the location of the aqueduct line and an office staff was organized for the necessary work on design. The location survey work, which was pushed to rapid completion by the end of February, 1914, included some 380 sq.mi. of topography, 362 mi. of transit lines, 1317 mi. of levels, 95 mi. of precise levels, 11,544 ft. of soundings at Indian Bay and 3897 ft. of test borings along the line with 7445 ft. at Indian Bay.

CONSTRUCTION RAILWAY—The located line of the Aqueduct runs south of the Canadian Pacific and National Transcontinental Rys. and North of the Canadi Northern Port Arthur Line, through an undeveloped s tion of the country. In order to bring in the supplies a machinery required for the aqueduct construction it w necessary to build a railroad along the aqueduct right- way. This road is of standard gage, with 60-lb. rails a substantial roadbed. Including yards, spurs to gravel p and sidings, there will be in all about 102.4 mi. of tra The maximum grade is 0.5 ft. per 100 ft., and the stan ard minimum curvature is four degrees. The contractor working tracks will be located between the railway and the trench, from which the track is distant 110 ft. The railway will cost approximately $1,200,000, and is being built by the Northern Construction Co., of Winnipeg. Tracklaying was completed on Dec. 17, 1914. There still remains some seven miles to be ballasted. This will be carried out early in the spring in order to have the road in shape for the main aqueduct work, which will be commenced as soon as the frost leaves the ground.

The clearing of the right-of-way was done by E. J. Bawlf, of Winnipeg. This work cost $79,360, and comprised: Clearing, 2586 acres; salvage, 7900 cords of wood, 349,000 lin.ft. poles, 14,467 fence posts.

A telephone line has been completed by the district from Winnipeg to Indian Bay. Residences have been erected for the engineers. Offtake drainage ditches have been dug. Roads have been built and other small works have been carried forward to prepare for the aqueduct construction.

FALCON RIVER DIVERSION DIKE AND CANAL—The water of the Falcon River, which drains about 50 sq.mi. of muskeg area into Indian Bay, is dark-colored and requires some time to clear itself through the natural agencies of the sun, wind and waves. In order to give the elements time to act on this water before it could possibly reach the intake a dike and canal are being built to divert the Falcon River into Snowshoe Bay.

Tomlinson & Fleming, who have the contract for the dike, which is some 7500 ft. long and which contains some 220,000 cu.yd., have practically finished this work. In depositing the materials in the dike these contractors made use of dinky engines, 4-yd. Western side-dump cars, a steam shovel and a scow. The method of depositing the material for the core or main body of the dike between the scow and the end of the dump, and running out the empty cars on the scow, proved rapid and economical (see Fig. 1).

*Information from the office of the Commissioners of the Greater Winnipeg Water District, Manitoba.

The tenders for the canal, which will be some 3700 ft. in length and require 37,000 cu.yd. of excavation, have been opened and an award was to be made on Jan. 15, 1915. An award was to be made on the same date on tenders for five railway water tanks.

CONCRETE AQUEDUCT—The aqueduct, 85.04 mi. in length, has been designed for a minimum rated capacity of 85,000,000 Imp. gal. per day. It will be built of concrete and will be reinforced where under pressure or at other required points. The standard sections are arch or horseshoe shaped for the gravity flow portions and circular for the parts of the line under pressure. (See Figs. 2 and 3 for views of test sections). There will be five depressed sections at river crossings, eight blowoffs and weirs, and two Venturi meters. The average gradient is 0.62 ft. per 1000, the lightest 0.11 ft. and the heaviest gradient is 1.5 ft. per 1000 ft.

Tables I to III give a variety of information regarding the dimensions, contractors, and quantities for the aqueduct.

TABLE I. LENGTH, SIZE AND SLOPE OF WINNIPEG AQUEDUCT, FROM LOWER TO UPPER END

Length		Size Ft. and In. Aqueduct Section	Slope Ft. Per 1000 Ft.
Ft.	Mi.		
24,140	4.572	8-0 diam.	0.780
33,400	6.326	8-9x7-4⅝	0.300
48,400	9.167	6-4¾x5-4¾	1.537
36,200	6.856	8-3¾x7-0	0.382
14,000	2.651	7-6 diam.[1]	Depressed section
43,598	8.257	7-4x6-2¼	0.744
30,000	5.682	7-7½x6-5¼	0.600
76,032	14.400	7-5½x6-3½[2]	0.684
10,000	1.894	6-7x5-6½	1.290
24,547	4.649	7-11½x6-8½[3]	0.480
53,083	10.054	8-9x7-4⅝[4]	0.300
10,900	2.064	8-10¼x7-5⅝	0.279
11,100	2.102	10-9x9-0	0.279
33,750	6.392	10-9x9-0	0.110
449,150	85.066		

[1]Depressed section at Brokenhead River Crossing.
[2]Depressed section at Whitemouth River Crossing.
[3]Depressed section at Birch River Crossing.
[4]Depressed section at Bogg River Crossings 1 and 2.
There is a Venturi meter at each end of the aqueduct.
The total amount of concrete in the whole aqueduct is 347,519 cu.yd., including 5085 cu.yd. in culverts, etc.

The cement, sand and gravel will be furnished by the district. At present tests are being made relative to the best aggregate available from the various gravel pits owned and leased by the district. The sand and gravel will be excavated from the district's gravel pits, will be put through a screening and crushing plant, screened to several sizes and remixed, after which it will be hauled to the work by daily supply trains operated by the district or under contract for the district.

CONTRACTS TO BE LET—Four seasons will be required to complete the work on the aqueduct contracts.

While the largest contracts have been let on this work there still remains considerable work for which bids will be asked in due time. This work, with the cost of the several items, as estimated in the report of the consulting engineer, is as follows:

TABLE II. CONTRACTORS FOR THE WINNIPEG AQUEDUCT

	Contract No.	Contract Section	Miles	Amount
J. H. Tremblay Co., Ltd.	30	Prairie	20.06	$945,945
Thomas Kelly & Sons	31	Brokenhead	17.76	1,301,485
Northern Construction Co. and Carter-Halls-Adlinger Co.	32	Whitemouth	18.20	1,268,680
	33	Birch	16.09	1,137,010
	34	Summit	12.92	1,489,520
Total				$6,142,640

TABLE III. PRINCIPAL QUANTITIES IN THE WINNIPEG AQUEDUCT

Excavation:		
Topsoil	171,000 yd.	
Earth	1,988,000 yd.	
Rock	20,300 yd.	
		2,179,300 yd.
Foundation, embankment and refill		2,195,000 yd.
Concrete:		
Aqueduct	342,434 yd.	
Culverts, etc.	5,085 yd.	
		347,519 yd.
Lumber		12,475 M. F. B. M.
Reinforcing steel		5,344,000 lb.
Cast iron and steel		71,000 lb.
Cast-iron pipe		29,500 lb.
Cast-iron gates		185,200 lb.
Copper expansion joints (approximate weight)		350,000 lb.
Cement (not included in the above contracts)		500,000 bbl.
		Cost
Intake at Indian Bay		$50,000
60-in. steel pipe line from Transcona to Red River		896,400
Tunnel under Red River		129,900
48-in. cast-iron pipe line from Red River to McPhillip's Reservoir		278,200
		$1,354,[illegible]

In addition to these, bids were received on Jan. 20, 1915, for gravel screening and crushing-plant equipment and for rolling stock for the railway. Bids for the cement supply for 1915, an engine shed at Deacon and miscellaneous buildings will be advertised in the near future.

Arrangements have been made through the Bank of Montreal, fiscal agents for the district, to provide some $4,000,000—the funds necessary for the work completed, the work in hand and that to be carried out. Certain financial arrangements have also been made with a view to the requirements of the succeeding years.

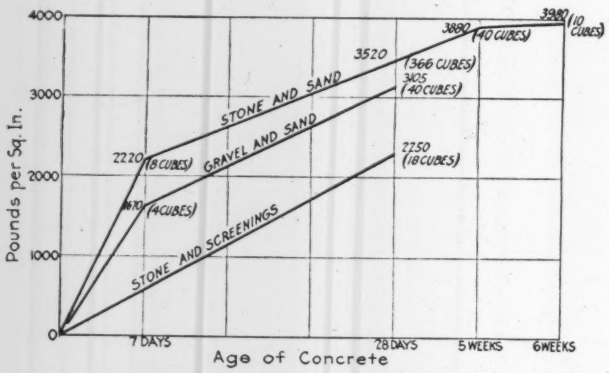
FIGS. 2 AND 3. ARCHED AND CIRCULAR TEST SECTIONS OF THE GREATER WINNIPEG AQUEDUCT

Some Field and Laboratory Tests of Concrete*

By H. S. Mattimore†

One of the important checks instituted within the past year on the concrete work of the New York State Highway Commission was in the testing of the finished product. Engineers in charge of concrete work are required to make 6-in. cubes from the mixed concrete as deposited in the work. Two cubes are taken from every 500 cu.yd. of concrete laid; this in the case of a concrete highway 16 ft. wide and of our standard thickness represents about 1700 lin.ft. of roadway. These specimens are stored in moist sand near the highway for 21 days at which time they are sent to the laboratory, where at 28 days they are tested for compressive strength.

Some of the tests so made are tabulated below. Table I is the record of some very high compression breaks



Curves Showing Relative Compressive Strength of 1 : 1½ : 3 Concrete Using Different Aggregates

(6-in. cubes made in field from concrete being used in highway work and stored in field 21 days)

obtained on 6-in. cubes made in the field from material being placed on the highway. There is some slight variation in the age of the different cubes due to delay in shipping, but a large proportion of the material is from 28 to 30 days old. This is not an average highway; in fact, it is one of the best. The two interesting features of these tests are the consistent uniformity of the breaks and the fact that the compressions are the highest I have ever seen recorded on plain concrete. I wish to call attention to the tests on the two sands from different banks, used in this concrete. Both are coarse sands with voids slightly below the average, they are comparatively clean and show a good compressive strength in mortar. They would not be called sharp sands as many of the grains are rounded. This material is from an old lake deposit and unfortunately rather limited.

It will be noted the last two cubes tabulated were mixed with stone screenings substituted for sand. This concrete was not used in the highway, but was made for experimental purposes only. It may be rather an unfair comparison as only two cubes were made from this material, but there is such a great difference in compressive strength that we do feel thankful that sand rather than stone screenings was used in the concrete placed in the highway.

TABLE I. COMPRESSION TESTS ON 1 : 1½ : 3 CONCRETE CUBES MADE FROM MATERIAL BEING PLACED ON N. Y. STATE HIGHWAY

Age	Break Lb. Per Sq. In., Aver. of 2 Cubes	Sand Used	Age	Break Lb. Per Sq. In., Aver. of 2 Cubes	Sand Used
30	5480+	No. 1 sand used	33	5555+	No. 2 sand used
34	4365		28	4500	
45	4815		28	4680	
36	3980		28	4050	
28	4810		30	4665	
29	4925		30	4420	
30	4675		28	5030	
28	4625		28	4450	
28	4875+		32	4450	
			33	1945*	

* The concrete represented by the last two cubes was made of stone screenings substituted for sand. It was made for experiment, not used on highway.

TESTS OF SAND USED IN ABOVE CONCRETE

No. 1. Voids = 31.2% Gradation %		Loam = 1.5% Compression Break	No. 2. Voids = 28.6% Gradation %		Loam = 2.5% Compression Break
Passing ¼ in.	100.0		Passing ¼ in.	100.0	
Passing No. 6,	93.3	1 : 3 Mortar	Passing No. 6,	94.0	1 : 3 Mortar
Passing No. 20,	41.7	Ottawa, 1420	Passing No. 20,	28.7	Ottawa, 1520
Passing No. 50,	10.5	Natural, 1987	Passing No. 50,	9.1	Natural, 1750
Passing No. 100,	1.8	Washed, 1802	Passing No. 100,	1.6	Washed, 1685
Passing No. 200,	0.9		Passing No. 200,	1.1	

The curves shown in the accompanying diagram give a good illustration of concrete obtained in actual practice. I believe this is a good, fair comparison of stone and gravel concrete, as both of these aggregates were of high grade, particular attention being paid to obtain a clean, uniform product passing all requirements for this class of concrete.

The material made with screenings, although showing a fair compression strength, is much below that obtained with a good sand.

In Table II are given other comparative results of the compressive strength of concrete with different kinds of aggregate.

TABLE II. COMPARATIVE TESTS ON 1 : 2½ : 5 CONCRETE USING DIFFERENT AGGREGATES

(6-in. Cubes)

Aggregates	Age	No. of Cubes	Compression Break, Lb. per Sq.In.
Slag and sand	28 Days	16	2000
Stone and sand	28 Days	134	1990
Gravel and sand	28 Days	142	1895
Stone and screenings	28 Days	28	1740
Coated gravel and sand unwashed	28 Days	48	1170

Table III is given to show the importance of proper sampling of sand sources. Both of these sands were from the same bank, being sampled by different engineers. Sand No. 1 is well within our specifications for use in the best grade of concrete while sand No. 2 is of a poor grade for any concrete.

TABLE III. TABLE TO SHOW EFFECT OF SAND GRADATION ON COMPRESSIVE STRENGTH OF CONCRETE

Gradation	No. 1	No. 2
Passing ¼ in.	100.0%	100.0%
Passing No. 6	94.1	74.5
Passing No. 20	38.4	30.4
Passing No. 50	14.4	26.7
Passing No. 100	2.0	8.4
Passing No. 200	1.4	4.5
Voids %	30.9	25.5
Loam %	3.6	4.0
Comparative Strength (lb. per sq.in.)		
Ottawa	1445	1975
Natural	2275	875
Natural Ottawa in %	157.5	44.2

Paving Work by Municipal Day Labor Instead of By Contract is urged by the Mayor of Columbus, Ohio, in his report for 1914. The new city charter will permit this step to be taken. It is stated that the city paving work amounts to about $600,000 a year and that contractors make 10% to 15% profit. Hence the city could save $60,000 to $90,000 a year—provided it could develop the same efficiency among employees.

*From a paper entitled "Testing and Handling of Aggregates for Concrete," presented to the Association of American Portland Cement Manufacturers.

†First Asst. Engineer, N. Y. State Highway Commission, Albany, N. Y.

Welding the Joints of Steel Gas Mains

Among the notable developments in mechanical engineering of the past half dozen years has been the rapid extension of the oxyacetylene process of welding and its application to new uses.

One of these recent applications is the welding together of lengths of wrought-iron or steel pipe to make a long continuous line, thus eliminating either screwed or bolted joints. Such welded pipes have recently been put down as gas mains in Chicago some eleven miles in extent, the diameter of the mains thus welded varying from 4 in. to 16 in. Similar work has been done at Philadelphia, and at the Panama-Pacific Exposition pipes welded in this way have been used for the entire gas-main system, comprising ten miles of pipes of 2-in. to 16-in. diameter. The smaller pipes also for gas distribution inside the Exposition buildings have been welded into continuous lengths.

FIG. 1. WELDING 12-IN. PIPE FOR GAS MAINS IN THE STREET

(The two men with the levers are turning the pipe as it is welded. The trench is on the opposite side of the street)

The accompanying photographs show the process of welding up lengths of gas main and lowering it into the trench.

The lengths of pipe are deposited along the street or on skids over the trench, and successive lengths are butted together. The pipe undergoing welding is turned gradually during the operation so that the welding is all done at the top, as shown in Fig. 1. Where large pipe is welded on skids over a trench, two welders often work on opposite sides of a joint.

In city streets, the mains laid on the street surface are welded usually to form lengths of 200 to 400 ft., as it is not desirable to open up very long trenches. These lengths are lowered into the trench by rope or chain slings, as shown in Fig. 2.

The question of contraction and expansion with temperature changes has of course to receive attention; but when a gas main is once buried in the trench the variation in temperature between summer and winter is probably not over 25°. To bring the pipe to a uniform temperature, the trench is partly filled with earth over the pipe leaving the joints at the abutting ends uncovered. A chamber is excavated around this, enabling the welder to work all around the pipe. If the ends draw apart as the pipe cools and contracts, the welder can apply additional metal to build up the joint.

Such welding is of course more difficult than welding the joints while the pipe is above ground. For this reason in open country very long lengths are welded up before lowering into the trench; 1000-ft. lengths are common and we are informed that at one place 100 lengths of 8-in. pipe, each 40 ft. long, were welded into a continuous section of 4000 ft. before being lowered into the trench. The lengths of pipe are of course capped and tested after welding the joints and before lowering into the trench.

FIG. 2. LOWERING A 160-FT. SECTION OF WELDED GAS MAIN INTO THE TRENCH

The welded joint is claimed to have usually from 80 to 90% the strength of the solid pipe, but can be made even stronger than the pipe by building up additional metal. The life of the pipe should be greatly increased since the thickness of the pipe is not reduced by cutting the threads upon it. The lengths of pipe to be welded

COST OF WELDING PIPE JOINTS AND YS

	6-In. Pipe		16-In. Pipe		8x6-In. Y— Cutting		Welding	
Labor, 30c. per hr.	20 min.	10c.	90 min.	$0.45	3 min.	1.5c.	22 min.	11c.
Oxygen, 2c. per cu.ft.	10 ft.	20c.	40 ft.	0.80	3 ft.	6.0c.	12 ft.	24c.
Acetylene, 2c. per cu. ft.	9 ft.	18c.	36 ft.	0.72	1 ft.	2.0c.	10 ft.	20c.
Filling wire, 12c. per lb.	¾ lb.	9c.	2 lb.	0.24	....	....	1 lb.	12c.
Total		57c.		$2.21		9.5c.		67c.

are made preferably with beveled ends, new metal being built up in the V-shaped opening at the joint, but square-end pipe can be welded if necessary. The use of the process is leading to the adoption of pipes 40 ft. long instead of the usual 20-ft. length.

Under ordinary conditions a skillful operator can weld in an hour about one joint on 12-in. pipe and from three to five joints on 4-in. pipe. The cost is said to be from 25 to 40% less than that of a recessed screw joint, including the cost of the coupling and its application. The cost of the joints is stated in the table on p. 233.

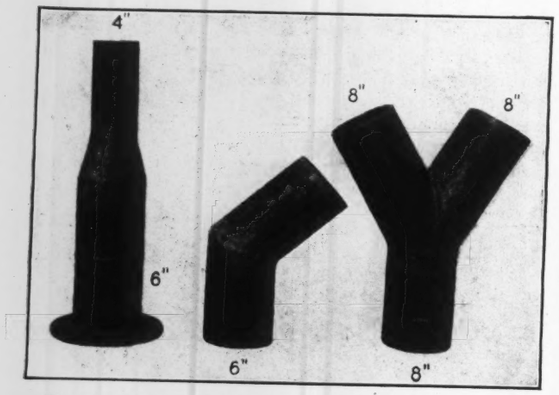


FIG. 3. WELDED SPECIALS FOR USE ON WELDED GAS MAINS

With the welded pipe, the branches, laterals, drips and various other fittings are made integral parts of the continuous main, while with screw-joint pipe they are separate and special parts whose numerous joints are often a source of trouble. Laterals are inserted at any point by cutting a hole in the main (with the cutting blowpipe) and welding in the end of the lateral. Fig. 3 shows a 6-in. angle, an 8-in. Y and a 6- and 4-in. reducer, all made by cutting the main and the smaller pipe to the desired shape with the torch and then welding the parts. The only material required to make up these specials are odd lengths of pipe of the required sizes, which can be cut and connected at any point and in any way. The cost of making the Y, with two 8-in. pipes connecting to an 8-in. main, is about 76c., as given in the table.

A great advantage of such continuously welded mains is that leaks from the joints, always a large source of loss in every gas distribution system, is wholly prevented. Thus these mains are especially advantageous for natural gas and oil pipe lines as well as for city gas distribution. For ammonia, and other refrigeration systems, elimination of leakage is important for safety as well as economy.

Much broader applications of the welding process to a wide range of piping and tubing seems reasonable to expect in the future as the number of skilled operators available increase and engineers become familiar with the process and its possibilities.

The apparatus used for the work shown in the illustrations is that of the Oxweld Acetylene Co., of Chicago. It consists of two cylinders of compressed oxygen and acetylene gas mounted on a two-wheeled truck and fitted with hose, regulators and the welding torch. The Peoples Gas light & Coke Co., of Chicago, has 26 of these outfits, which are used for making repairs to machine etc. at the gas works as well as for welding the mains.

Operation of the Plainfield Sewage-Works

BY JOHN R. DOWNES*

The accompanying notes on the operation in 1914 of the septic tank and contact beds which comprise the sewage works of Plainfield, N. J., are submitted in view of the historic interest of the works. It will be remembered that this plant has been considerably overloaded for several years and has been kept in operation only by everlasting vigilance.

The weakest part of the plant is the old, shallow, one-story settling tank, which after a few months' service disgorges large volumes of suspended solids, causing a serious mechanical interference with the working of the contact beds. With local arrangements, frequent cleaning of the tanks is impossible. The extent of this interference is by no means measured by the weight of the suspended solids escaping from the dirty tanks. The solids are flocculent in character and of low gravity. Their volume increases many times in proportion to their weight at times of disgorging.

As a partial offset to the unfavorable condition incident to poor settling arrangements, strainers of ⅜-in. cinders are provided to remove the coarsest of the suspended solids. These strainers, 1 ft. deep and 10 ft. wide, are countersunk, as it were, in the main filling material of the primary contact beds and extend along the two outer edges of each bed. The strainers are especially efficient in preventing any scum formation on the coarse stone of the primary beds, from which its removal is almost impossible, and, to say the least, far from economical.

From the strainers, the collected solids, together with ½ in. or so of the cinders, can readily be skimmed. The contact beds proper present the appearance of newly crushed stone, and, being free from scum, are also free from odors, as well as from the possibility of sepsis.

The cinders are used repeatedly, after being washed on a revolving screen while a spray of water is played over them. The screen is revolved and the water is pumped by a 3½-hp. gasoline engine. The cost of fuel, oil and labor for cleaning is 9c. per cu.yd. of clean cinders. One hundred parts of the dirty cinders yield about 80 parts of clean cinders and 40 parts sludge in the form of a thick paste from which no superfluous water can be drained. The clean cinders have about 35% voids. The sludge adheres closely to the cinders and appears very compact. It does not resume this compact form (almost like asbestos board) after separation from the cinders.

The cost of extra labor, under local conditions, for the operation of the cinder strainers is 44c. per million gallons of sewage treated. This extra cost should really be charged against the inefficiency of the settling tanks and will be offset by the extended working life of the primary contact beds. It is hoped that this extended life will reach a 33% increase. It will prove economical if it reaches 11%. Entirely aside from this extension of life,

*Chemist and Manager, Sewage Works, 48 Rock Ave., Plainfield, N. J.

MONTHLY AVERAGES OF ANALYSES OF SEWAGE-WORKS OF PLAINFIELD, N. J., DECEMBER, 1913, TO NOVEMBER, 1914*

(Parts per 1,000,000)

Month	Temperature Air Max.	Temperature Air Min.	Temperature Septic Effluent	Suspended Matter Screened Sewage	Suspended Matter Effluent from Septic Tank	Suspended Matter Effluent from Primary Bed	Suspended Matter Effluent from Secondary Bed	Oxygen Consumed Screened Sewage	Oxygen Consumed Effluent from Septic Tank	Oxygen Consumed Effluent from Primary Bed	Oxygen Consumed Effluent from Secondary Bed	Nitrates as NO_3 Primary Bed	Nitrates as NO_3 Secondary Bed	Nitrates as NO_2 Primary Bed	Nitrates as NO_2 Secondary Bed	Putrescibility: Per Cent Samples Stable 14 Days
D[illegible]	47	28	58	163	66	23	8	122	91	45	21	1.1	4.7	0.11	0.16	90.0
Ja[illegible]	38	25	52	133	63	21	7	111	94	42	23	0.9	3.1	0.11	0.11	95.5
F[illegible]	38	17	50	113	54	22	9	99	83	39	23	1.1	2.3	0.08	0.03	88.3
Mar[illegible]	45	29	50	102	60	22	8	104	94	45	22	0.7	1.8	0.05	0.04	88.9
Apr[illegible]	60	39	52	157	79	29	13	132	107	55	33	0.9	1.9	0.09	0.06	81.9
May[illegible]	71	53	58	130	47	27	16	111	83	52	32	0.4	3.0	0.05	0.12	56.3
June[illegible]	82	58	66	155	54	32	23	113	90	51	35	0.2	1.9	0.02	0.10	36.8
July[illegible]	86	65	68	156	51	28	17	124	93	53	37	0.2	2.6	0.03	0.07	68.1
Aug[illegible]	85	65	68	136	80	28	12	119	102	50	32	0.3	3.0	0.02	0.04	78.4
Sept[illegible]	76	52	67	165	75	27	15	119	93	50	28	0.4	2.7	0.12	0.15	78.2
Oct[illegible]	68	48	66	192	70	31	16	126	102	47	32	0.7	2.6	0.21	0.15	50.0
Nov[illegible]	54	34	60	155	43	21	10	138	96	51	30	1.3	3.9	0.17	0.17	100.0
Average	62	45	60	146	62	26	13	110	94	48	29	0.7	2.8	0.09	0.10	

* The septic tanks were cleaned in December, 1913, and at the end of October, 1914.

however, the strainers seem to be the best solution of the scum and odor problem of the primary beds.

The practice of removing scum and dirty cinders from the secondary beds has been abandoned. The beds are so graded as to minimize the pooling of water on the surface. The first sign of scum at any damp place is at once broken up by raking. Scum, odors and sepsis are thus eliminated, as in the case of the primary beds. The beds are in a clean, healthy aerobic condition, some of them after 14 years of service.

Under the previous system of allowing the scum to form and then removing it, a large part of the volume of the beds had reached a septic condition. Under the newer method of treatment, instituted nearly two years ago, all signs of septic conditions permanently disappeared within a few weeks.

Screenings at the plant amount to 4.3 cu.ft., sludge retained in the tanks amounts to 2.5 cu.yd., and solids collected on the strainers to 0.3 cu.yd. per million gallons of sewage treated.

The accompanying table summarizes the analysis of screened sewage and of effluent from the tanks and from the primary and secondary contact beds made during the year.

Annual Meeting of the Canadian Society of Civil Engineers

The twenty-ninth annual meeting of the Canadian Society of Civil Engineers was held in Montreal, Jan. 26 to 28. In addition to the regular business, there were excursions to such engineering works as the Cedars power development, Angus shops of the Canadian Pacific Ry., and the Mount Royal tunnel of the Canadian Northern Ry. The entertainment features included a lunch, a smoker, and the annual dinner.

MEMBERSHIP—The report of the council showed a membership of 3059, and an increase of 265 for the year. Report showed that the eight old branches were in flourishing condition and that a new branch had been established at Edmonton. In British Columbia, under a provincial water act, members of the society have secured special recognition by the government of the province. Similar recognition is being sought from the government of Saskatchewan.

OFFICERS—The new President of the society is Francis Clarke Campbell, Chief Engineer of the Department of Railways for the Province of British Columbia. The following Vice-presidents were elected: A. St. Laurent, of Ottawa, and E. E. Brydone-Jack, of Winnipeg. As Councillors, A. Surveyer, of Montreal, C. B. Brown, of Moncton, T. A. J. Forrester, of Quebec, A. A. Dion, of Ottawa, J. L. Weller, of St. Catherines, W. G. Chace, of Winnipeg, and M. J. Ker, of Vancouver, were elected.

SPECIAL REPORTS—The address of the President, M. J. Butler, C. M. G., departed from the practice of his predecessors, who have discussed special technical problems in their annual addresses. He gave a review of the industrial and chemical developments of the Dominion during the last decade.

The Committee on Conservation reported that the collation and publication of gage records by the Dominion Commission on Conservation was still in its preliminary stage. Other activities of the Conservation Commission were discussed—forest protection, establishment of a uniform datum plane for the Dominion, and the plan of J. B. Challies for a central bureau for the distribution of public documents issued by different departments, and for making available to each department the technical investigations of all the others.

A set of guide specifications were adopted as prepared by a Committee on Concrete and Reinforced Concrete (W. J. Francis, of Montreal, Chairman). These general specifications were grouped under materials, methods of calculation, unit stresses, general requirements in design, workmanship, tests and inspection.

The report of the Committee on General Clauses for Specifications (Henry Holgate, of Montreal, Chairman) was intended to serve a similar purpose. These clauses were grouped under definitions; duties of engineers; certificates and payments; plans and drawings; claims for extra work; carrying on work; contractors' responsibility; contractor in default, and miscellaneous. While the report as a whole was received with much appreciation, yet there were some criticism of a few parts which seemed to be too severe on the contractor. These paragraphs, it was held, would not stimulate that spirit of coöperation between the engineer, the purchaser, and the contractor which was necessary to lowest prices. Some of the particular points involved were: responsibility for engineers' mistakes, compensation for delays not due to the contractor, reliance on the engineers' opinions, right of the engineer to change the contract price, etc.

The Committee on Rails (H. G. Kelly, of Montreal, Chairman) recommended the adoption of the rail specifications of the American Railway Engineering Association.

The Committee on Track (H. R. Safford, of Montreal, Chairman) made a progress report on the subjects of size and spacing of ties, and proper method of measuring efficiency of track forces. They showed three programs for the study of the two subjects by securing data from members.

News of the Engineering World

Panama Celebration Postponed

A postponement of the celebration of the Panama Canal completion was officially announced at Washington some days ago. It was stated that the slides in the vicinity of Gold Hill continued to obstruct the canal prism so that while ordinary traffic is passing through the canal without difficulty, the dredges will not be able to clear the channel to its full depth and width at the points where the slides have occurred until after the date in March originally set for the canal's official opening, when it was intended to have a battleship parade headed by the "Oregon," carrying, among others, the President on his way to the opening of the Panama-Pacific Exposition at San Francisco. It is now announced that the President will make the trip overland.

Increasing Safety of Rapid Transit in New York City

Since the recent burning out of a manhole full of feeder cables supplying the New York Subway, with the consequent tying up of traffic for nearly a whole day and the overcoming by smoke of some 200 passengers in stalled trains (as noted in ENGINEERING NEWS, Jan. 6, 1915), the Public Service Commission's Engineering Department has been investigating the safety precautions that might be increased on this system. Although the accidents have been notably few* there has been a public demand for prevention of the discomfort and danger of mishaps like this last one.

On the report of Alfred Craven, Chief Engineer of the Commission, the following definite recommendations are presented.

(1) The addition to the company's present private telephone system of a second system entirely distinct and properly separated from the power cables. (2) The installation of an additional lighting system from an outside source, and rearrangement of the present lighting system, bringing in current for one side of the subway from the 59th St. power plant and for the opposite side from the 74th St. plant, and having these two supplies so arranged that in case of a breakdown of one plant the entire load may be switched to the other. (3) Provision for a separate source of power supply for the ventilating fans to be installed so as to be automatically available in case the power from the substations fails. (4) To prevent smoke from entering the subway in cases similar to the last accident, ventilation provided from the manholes to the surface of the street so that smoke and gases will escape through these openings. (5) A thorough and complete segregation of high- and low-tension cables and lighting and signal cables. (6) Increase of emergency exit facilities through the ventilating chambers to the street.

Nothing is reported on the training of the subway employees for better handling of unusual situations, though Mr. Craven states that "other recommendations are still under consideration."

C. W. Wilder, Electrical Engineer of the Commission, has reported that it is possible to construct an all-steel car which would not weigh more than the heaviest type of wooden car now operated upon the elevated railroads in Manhattan and the Bronx. This car would weigh about 24,000 lb. This report is a part of a general scrutiny of conditions on the elevated lines.

*The Interborough Rapid Transit Co. has widely advertised that only one person in 842,620,000 carried has been killed in the subway. This probably means in train accidents of five years' operation, for the 1912 report of the Public Service Commission for the First District of New York shows 37 killed out of 607,244,697 carried that year on all the Interborough lines, including the elevated roads.

A Waterproofing Patent Decision

The United States Circuit Court of Appeals, Seventh District, on Jan. 4, 1915, affirmed the decision of a lower court which upheld Patent No. 851,247 issued on April 23, 1907 to S. B. Newberry (assigned to the Medusa Concrete Waterproofing Co.) for a waterproofing for concrete, and adjudged the McCormick Waterproof Portland Cement Co. and Sidney T. Sjoberg to be infringers of the patent. Justice Baker, in rendering the decision, in effect declared that the Newberry patent covered all forms of concrete waterproofing where the compound contains "the insoluble lime salt of a fatty acid substantially free from glycerin or other soluble substance."

Projected Sewerage and Sewage Disposal Works for Allentown, Penn.

The sewerage system with disposal by Imhoff tanks, sprinkling filters and secondary settling basins now proposed for Allentown, Penn., is of more than usual interest in view of the fact that although this city had a population of 51,913 in the year 1910, it is still without a sanitary sewerage system. A small area of the city has combined sewers with which there are a few house connections.

In March, 1908, Chas. D. Weirbach, city engineer, in conjunction with Hering & Fuller, consulting engineers of New York City, prepared plans and specifications for sewerage and sewage treatment. These plans were approved by the State Department of Health in July, 1908. It was proposed to raise a part of the cost of the project by assessments for benefits and a part of it by a bond issue. In the spring of 1909 a proposed bond issue of $400,000 was voted down by a popular vote of 2 to 1. Since then various proposals to revive the scheme have come to nothing. Recently Mr. Weirbach has been revising the plans to meet changed conditions and the city has appropriated $25,000 out of its general revenue towards purchasing a farm of 117 acres on which is located the disposal site recommended in 1908. This farm is now valued at $100,000, or more than double the price at which it might have been secured when the report was made.

The plans of 1908 provided for a collecting system about 62 miles in length, a high-level interceptor and outfall sewer, a pumping station, a force main about 4000 ft. long, a 2,000,000-gal. settling tank, 2½ acres of sprinkling filters, and a final settling basin. The esti-

mated cost in round numbers was $1,000,000. In the revised plans it is proposed to substitute an Imhoff tank for the original settling tank and to provide more sewers for the annexed districts. Otherwise no changes are proposed. No further work on the plans will be done until the land has been secured.

A Hydraulic Suction Dredge Was Wrecked in the storm off the north coast of Long Island, Feb. 2, and three of the crew were drowned. The dredge was anchored near the entrance of the harbor of Port Jefferson, N. Y., and was swamped during a severe northerly gale.

A 30-In. Water Main Burst at the corner of Eighth Ave. and 42d St., New York City, about 10:30 p.m., Feb. 2. A flood was caused north on Eighth Ave. and west on 42d St., because for several hours the Water Department men were unable to find the main which caused the trouble. The main was a very old one and was under only about 30 lb. pressure.

The Last Gap in the Canadian Northern Ry's new transcontinental line has been completed. The ceremony of driving the last spike will be held in the spring.

Oil Fuel Has Been Adopted for all the tugs and ferries operated by the Long Island R.R. in New York Harbor. It is reported that the railroad company expects to save $20,000 annually by the change from coal to crude oil.

A Bonus of $65,800 for completing the concrete work of the Cleveland water-filtration plant six months ahead of the contract time has been allowed the John F. Casey Co. of Pittsburgh, Penn., by the Cleveland Board of Control. The company claims that a total of $72,000 is due it but the city holds that no allowance should be made for Saturday half holidays.

The Boston Dry Dock cannot be built without further legislative authority although nearly $400,000 worth of preliminary work has been done and the contract for the dock itself has been awarded. This opinion has been given by Atty.-Gen. Attwill of Massachusetts and will probably result in prompt action by the legislature now in session.

An Attempt to Blow Up the Bridge carrying the Canadian Pacific Ry. across the St. Croix River between Maine and New Brunswick was made in the early morning of Feb. 2. The damage done was confined to the track on the Canadian side and was quickly repaired, train service being resumed some hours later. A man claiming to be a German officer was arrested at Vanceboro, Me., by the police there and is said to have confessed to the attempt. He is being held by the American authorities pending the settlement of the question of jurisdiction.

The Site Chosen for the Pumping Station of the new high-pressure fire protection system for Boston in Fort Point Channel near the South Terminal Station has been disapproved by the War Department as having "an undesirable effect upon the use and development of the channel." It is stated that this is at least the third site which has had to be given up. It is thought that the next choice will be a piece of land in the train yard of the South Station, across the street from the site just disapproved. The early securing of the site is now desirable since the work of laying the high-pressure pipe system is nearly completed.

Proposed Changes in the Alaska Railway Commission are opposed by the Board of Direction of the American Society of Civil Engineers, which at its meeting on Jan. 20 adopted the following resolutions:

Resolved that in the opinion of the Board of Direction of the American Society of Civil Engineers, it would be unfortunate for the present Alaska Railway Commission to be superseded and that the interests of the public demand that the present Commission be allowed to carry out the construction, as well as the location, of the line.

This resolution was forwarded to President Wilson on Jan. 22 in a letter signed by President Marx, as President of the American Society.

Improved Means of City Wastes Disposal at Wilmington, Del., have been recommended by the board of health in a report to the city council. The recommendations include: Household separation and city collection of garbage, ashes and combustible refuse; a high-temperature incinerator at the present crematory; the burning of garbage and combustible refuse; the recovery and sale of marketable metals, glass, etc.; the use of ashes and incombustible refuse for filling low land; the removal of all trades wastes to the incinerating plant by the producers of the wastes; and that all the work outlined be placed in one department, headed by a competent director or superintendent.

An Amendment to the Ohio Conservancy Law, authorizing the formation of districts for flood prevention and other purposes, has been introduced in the Ohio legislature. The important provisions of the amendment are: (1) That no reservoirs shall be built for flood prevention; (2) that there shall be one director from each county, to be elected by the people of the county as a whole for each conservancy district; (3) that where more than one county is involved the concurrence of two-thirds of the directors will be necessary in order to secure the adoption or the amendment of any plan for conservancy work. The original law provided that directors of conservancy districts would be appointed by judges of the county courts (see "Eng. News," Feb. 5, 1914, for an outline of the law and for editorial comment upon it).

The Sliding of an Earth Fill under a moving railway train is one of the rarest of the causes of railway accidents. Such an accident is recorded, however, in accident bulletin No. 52 of the Interstate Commerce Commission. It occurred May 27, 1914, near West Point, Tex., on the Missouri, Kansas & Texas Ry. The fill was made in a ravine at the base of a hill and was composed largely of a soapstone formation, which when wet had little holding qualities. Apparently no outlet was left through the fill for water draining from the hills behind, and a pool was formed back of the fill about 7½ ft. deep and 75 ft. long. During the three months preceding the accident, Weather Bureau records show that nearly 25 in. of rain had fallen. The track on the fill was on a 4° curve, and the train which was passing over the fill when the slide occurred was a slow freight running only 5 to 10 miles per hour. The locomotive and first four cars were derailed and the engineman and fireman were crushed under the overturned locomotive. The roadmaster, trainmaster, and district engineer agree that the fill had become soft from the continued rains. While the embankment slipped under a slow freight train, it is stated that a passenger train running 20 to 25 miles per hour had passed over the track only 40 minutes earlier.

The Municipal Asphalt Plant of New York City, which was completed and put in operation in May last, is the largest municipal asphalt plant in the United States. According to the annual report of Henry Welles Durham, Chief Engineer of the Bureau of Highways, Borough of Manhattan, the municipal plant laid during the last seven months of 1914 a total of 12,877 sq.yd. of asphalt paving at an average cost of 99.8c. per sq.yd., which cost included 18.4c. per sq.yd. for overhead and depreciation charges. The average price on 114,800 sq.yd. of asphalt paving laid by contract during 1914 was $1.47 per sq.yd. In operating the city plant, moreover, portland cement was used as a filler, where the private companies use limestone dust, the city plant used a grade of sand costing 80c. per cu.yd., where the paving companies used sand costing 50c. The city also used broken stone costing $1.10 per cu.yd., where the companies use a grade costing $1.00 or less. It is expected that the better grade of materials will result in greater durability in the paving laid by the city's plant. The cost of constructing the municipal plant was $153,000 and during the seven months it has been in operation the above figures show a direct saving to the city of $61,626, besides securing promptness in making repairs and other gains described in Mr. Durham's reports.

Selling Undrained Lands in the Florida Everglades in a fraudulent manner through the United States mails was the charge on which four officers of the Chambers Land Co. were recently convicted in the Federal Court at Kansas City and sentenced to prison terms of one to ten years. This was one of the Everglades land schemes of which a great deal was heard in Washington two or three years ago. As some of our readers will recall, an investigation was made by the Federal Bureau of Drainage Investigations with reference to the projects for draining the Everglades; and as a result of this investigation a pamphlet was prepared for publication putting investors on their guard against the purchase of Everglades land without careful investigation. Prominent state officials of Florida and representatives of that state at Washington, however, strongly opposed any attempt to interfere with the sales of Everglades lands and had sufficient influence to cause the suppression of the pamphlet and the dismissal and indictment on trumped up charges of the Chief of the Bureau of Drainage Investigations, C. G. Elliott, and his principal assistant. The campaign waged in defense of Mr. Elliott, however, by "Engineering News" and by engineering societies and prominent members of the profession resulted in his complete exoneration. The conviction and sentence of some of those engaged in the Everglades land fraud, who were three years ago prominent in the persecution of Mr. Elliott, furnishes further justification, if justification were needed, of the work done by the profession for his defense. It is authoritatively stated that the post office authorities are proceeding with the prosecution of other operators in Everglades lands for fraudulent use of the mails.

PERSONALS

Mr. Irving L. Ryder, San Jose, Calif., has been appointed County Surveyor of Santa Clara County, Calif.

Mr. Joseph B. Eastman has been appointed a member of the Massachusetts Public Service Commission, to fill the vacancy caused by the resignation of Mr. George W. Anderson.

Mr. R. Y. Walker, of Belton, Tex., has been appointed Engineer in charge of road work in the Holland district, to fill the unexpired term of Mr. E. A. Kingsley, Assoc. M. Am. Soc. C. E., resigned.

Mr. Albert C. Lehman has been elected President of the Blaw Steel Construction Co., Pittsburgh, Penn. Mr. Lehman has been Vice-President and General Manager of the company since its organization.

Mr. O. H. Tittmann, M. Am. Soc. C. E., Superintendent of the United States Coast & Geodetic Survey, Washington, D. C., has been elected President of the National Geographic Society, to succeed the late Henry Gannett.

Mr. Walter Alexander, formerly District Master Mechanic of the Chicago, Milwaukee & St. Paul Ry., at Milwaukee, has been appointed a member of the Wisconsin Railroad Commission, succeeding Mr. David Harlow.

Mr. R. N. Hudson, General Manager of the Louisville & Atlantic R.R., at Versailles, Ky., has been elected President and General Manager of the Louisville, Henderson & St. Louis Ry., to succeed the late L. J. Irwin.

Messrs. C. R. Dodge and G. H. Edbrooke, formerly of Dodge & Dodge, New York, have opened an office at 33 W. 42d St., New York, and, under the firm name of C. R. Dodge & Co., will specialize in concrete handling equipment.

Mr. W. R. Dawson, Assistant Division Superintendent of the Norfolk & Western Ry., at Bluefield, W. Va., has been appointed Assistant to the General Manager, with office at Roanoke, Va. Mr. Dawson succeeds the late G. W. Merrell.

Mr. H. T. Reinecker, Roadmaster of the Norfolk & Western Ry., at Shenandoah, Va., has been appointed Assistant Superintendent of the Scioto Division with office at Portsmouth, Ohio. Mr. Reinecker succeeds Mr. L. C. Ayers, transferred.

Mr. L. C. Ayers, Assistant Division Superintendent of the Norfolk & Western Ry., at Portsmouth, Ohio, has been appointed Assistant Superintendent of the Pocahontas division, with office at Bluefield, W. Va., to succeed Mr. W. R. Dawson, promoted, as noted elsewhere.

Prof. Leon Arnal, who was appointed Professor of Architectural Design in the College of Engineering, University of Minnesota, at the beginning of the current year, is now serving as French interpreter with the 1st Connaught Rangers, British Indian Contingent Force.

Mr. Edward P. Burch, Fellow Am. Inst. E. E., 922 Plymouth Bldg., Minneapolis, Minn., has opened an office as Consulting Engineer in the Dime Bank Bldg., Detroit, Mich. Mr. Burch is a graduate of the College of Engineering of the University of Minnesota. Mr. Lester H. Sears is associated with Mr. Burch.

Mr. G. Edward Tilt, M. Am. Soc. C. E., has resigned his position with John Monks & Sons, New York, to engage in the general practice of engineering and contracting, specializing in harbor construction, marine terminals and warehouses, under the firm name of G. E. Tilt & Co., 90 West Broadway, New York City.

Mr. H. Bortin, Assoc. M. Am. Soc. C. E., formerly Engineer in Charge of the Valuation Department of the Union Pacific R.R., and lately Assistant to General Secretary of Presidents' Conference Committee on Federal Valuation of the Railroads, has opened an office at 149 Broadway, New York for private practice as a consulting valuation engineer.

Mr. Sheridan S. Kendall, senior member of the Colorado Public Service Commission, has been made chairman for one year, to succeed Mr. A. P. Anderson, who was chairman during the past year. Mr. M. H. Aylesworth has been appointed as the new member of the Commission. He will serve in the capacity of legal adviser to the Commission.

Messrs. Frederick J. Shollar and Frank A. Hersch, Altoona, Penn., have dissolved partnership, as stated in our issue of Jan. 28. Both members will continue the practice of architecture under their own names, Mr. Frank A. Hersch entering into partnership with his brother, Mr. Herman R. Hersch, under the firm name of Hersch & Hersch, Altoona, Penn.

Mr. Guy E. Marion, who has been in charge of the Information Department of Arthur D. Little, Inc., Boston, Mass., for the past five years, has resigned and will devote h time to the organization and development of special librarie or information departments, in business houses and industri plants, with offices at 1101-4 Tremont Bldg., Boston, Mas

Col. Harry F. Hodges, M. Am. Soc. C. E., former Vic Governor of the Panama Canal, has been designated Engineer Officer in Charge of the Washington River an Harbor District and of the Washington Aqueduct, with hea quarters at Washington, D. C., succeeding Col. Henry C. New comer, who will return to his former duties as a memb of the River and Harbor Board.

Mr. Beverly Reid Value, M. Am. Soc. C. E., who for severa years past has been connected with the Empire Engineerin Corporation and H. S. Kerbaugh, Inc., 6 Church St., New York is now associated with George W. Rogers & Co., Inc., 2 Broadway, New York. Mr. Value has had many years' ex perience as an engineer on New York subway work, th Niagara power tunnels and the Croton Aqueduct.

Mr. Clarence D. Pollock, M. Am. Soc. C. E., Park Row Bldg., New York, has been appointed Consulting Engineer, and Mr. Zenas W. Carter, Boston, Mass., Field Secretary, of the Granite Paving Block Manufacturers' Association of the United States. Mr. Pollock has served as Chief Engineer of Paving of the City of Havana, Cuba, and Chief Engineer of the Department of Paving of San Antonio, Tex. He was formerly connected with the Bureau of Highways of the Boroughs of Manhattan and Brooklyn, New York City.

Mr. John N. Carlisle has resigned as State Highway Commissioner of New York, at the request of Governor Whitman, who has appointed Mr. Edwin A. Duffy, a former District Atttorney of Cortland County, and a personal friend of the Governor, at the head of the Highway Department. In reply to Mr. Carlisle's letter of resignation, Governor Whitman wrote that he accepted it "with a full appreciation of what Mr. Carlisle had endeavored to accomplish in the Highway Department and of his unblemished personal reputation and unimpeachable character." Mr. Carlisle was appointed Highway Commissioner by Governor Sulzer in May, 1913. Soon after his appointment he effected an entire reorganization of the department upon nonpolitical principles, placing the heads of the nine state divisions under civil service, and the whole engineering bureau under Mr. George A. Ricker, M. Am. Soc. C. E., as First Deputy Commissioner.

Dr. Allen J. McLaughlin, the recently elected Health Commissioner of Massachusetts, has selected his administrative staff, and the appointments have been approved by the Governor. Under the reorganization of the State Health Department the appointments are as follows: Mr. X. H. Goodnough, M. Am. Soc. C. E., Chief Engineer, Director of the Division of Sanitary Engineering, with a salary of $5000 per annum; Mr. H. W. Clark, Chemist, Director of the Division of Water and Sewerage Laboratories, $4000 per annum. Mr. William C. Hanson, Director of the Division of Records and Accounts, $3000 per annum; Mr. Hermann C. Lythgoe, Director of the Division of Food and Drugs, $3000 per annum; Prof. Milton J. Rosenau, Pathologist, $1500 per annum; Dr. Eugene R. Kelley of Seattle, Director of Communicable Diseases, $4000 per annum. Messrs. Rosenau and Kelley are new members of the staff. The other three occupied similar offices under the old organization.

Mr. Ambrose Swasey, who has just given $200,000 to create a fund to be known as the Engineering Foundation, for the benefit of the engineering profession, is a partner in the famous firm of Warner & Swasey, of Cleveland, which has a world wide reputation for its achievements in the construction of astronomical instruments. Mr. Swasey was born on a farm in Exeter, N. H., and learned the machinist trade in his native town. In 1870 he entered the employ of the Pratt & Whitney Co. of Hartford, Conn., and quickly showed extraordinary ability in the solution of complex mechanical problems. He invented the epicycloidal milling machine for producing the true theoretical curves of gear teeth. In 1880 Mr. Swasey, in company with Mr. Worcester R. Warner, with whom he had been associated in work in his native town and who had gone with him to the Pratt & Whitney Works, removed to Cleveland and undertook the business of building high-class machine tools. The interest of both partners in science, especially astronomy, and the reputation for high-class workmanship which the new firm soon gained led them to undertake the construction of the highest class astronomical instruments. Only six years after the new firm was organized, they received a contract under severe international competition for constructing and mounting the great telescope at the Lick observatory in California, the greatest refracting telescope ever made at that time. Since that time the machinery and mountings for the largest telescopes constructed in the United States have been carried out by the Warner &

Swasey firm, in addition to their regular business as builders of machine tools. He is one of the most respected citizens of Cleveland. He has been prominent in philanthropic and educational work, has served as a director of some of the principal financial institutions, and has been President of the Cleveland Chamber of Commerce. He was a charter member of the American Society of Mechanical Engineers, and was President of the Society in 1904. A portrait and biography of Mr. Swasey were published in our issue of Jan. 28, 1904. He has always taken an active personal interest in the engineering societies of which he is a member and in every movement for the welfare of the profession. His notable gift for the benefit of engineers, made with characteristic modesty, will be appreciated by his many friends in the profession as a characteristic expression of Mr. Swasey's generosity.

OBITUARY

W. D. Scott, General Manager of the Spokane, Portland & Seattle Ry., Oregon Trunk Ry., Oregon Electric Ry. and United Rys. Co., died Jan. 21, at his home in Portland, Ore., at 56 years of age.

John M. Mack, a prominent contractor and financier, of Philadelphia, died on Jan. 26, aged 63 years. He began business as a paving contractor in Philadelphia in 1876, and became prominent in the asphalt paving field. He organized a dozen years ago the Asphalt Co. of America to consolidate all the asphalt interests of the United States. Afterward he engaged in street railway finance and organized the Philadelphia Rapid Transit Co.

Horatio F. Forrest, one of the earliest civil engineers of Manitoba, Canada, died in Winnipeg, Jan. 18, in his 76th year. He was born in Montreal, and began his professional career on the Intercolonial Ry. He went West in 1873, and was engaged in location surveys for the Canadian Pacific Ry. east of Winnipeg. For some years he filled the position of Government Inspector of railways, and was subsequently engaged as District Engineer of the Canadian Northern Ry., resigning that position about two years ago on account of failing health. Mr. Forrest left a widow, two sons and one daughter.

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN CONCRETE INSTITUTE.
Feb. 9-12. Convention at Chicago, Ill. Secy., E. E. Krauss, Harrison Bldg., Philadelphia, Penn.

CHICAGO CEMENT SHOW.
Feb. 10-17. At Chicago. Secy., J. P. Beck, Universal Portland Cement Co., Chicago.

AMERICAN CONCRETE PIPE ASSOCIATION.
Feb. 15-16. Annual convention at Chicago. Secy., E. S. Hanson, 538 South Clark St., Chicago, Ill.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Feb. 15-17. Annual meeting at New York City. Secy., Bradley Stoughton, Engineering Societies Building, New York City.

NATIONAL PAVING BRICK MANUFACTURERS ASSOCIATION.
Feb. 15-20. Annual convention at Detroit, Mich. Secy., Will P. Blair, Engineers Building, Cleveland, Ohio.

IOWA ENGINEERING SOCIETY.
Feb. 17-19. Annual meeting; at Iowa City. Secy., S. M. Woodward, Iowa City, Iowa.

MINNESOTA SURVEYORS AND ENGINEERS SOCIETY.
Feb. 19-20. Annual meeting, at St. Paul, Minn. Secy., George H. Herrold, City Hall, St. Paul, Minn.

INDIANA SANITARY AND WATER SUPPLY ASSOCIATION.
Feb. 24, 25. Annual meeting at Indianapolis, Ind. Secy., Dr. W. F. King, State Board of Health, Indianapolis, Ind.

ILLINOIS WATER SUPPLY ASSOCIATION.
Mar. 9-11. Annual convention at University of Illinois. Secy., Edward Barlow, Urbana, Ill.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.
Mar. 16-18. Annual meeting at Chicago, Ill. Secy., E. H. Fritch, 900 So. Michigan Ave., Chicago, Ill.

Louisiana Engineering Society — At the annual meeting held Jan. 9 the following officers were elected: President, L. C. Datz, Engineer of Roadway, New Orleans, La.; Secretary, W. T. Hogg; Treasurer E. H. Coleman.

Cornell Society of Civil Engineers—The statement under this head in "Engineering News," Jan. 28, that Charles N. Green was President, was an error. The President of the Society is Clyde Potts, Consulting Engineer, New York City.

Montana Institute of Municipal Engineers — The third annual meeting was held at Billings, Mont., Jan. 19 to 21. The following officers were elected: C. E. Durland, of Billings, President; Vice-President, C. W. Helmick, Helena; Secretary-Treasurer, C. C. Widener, Bozeman.

Forest Products Federation—This organization has arranged for a meeting in the interest of all branches of the lumber industry, to be held at the La Salle Hotel, Chicago, Feb. 24-25. E. A. Sterling is Secretary and R. S. Kellogg is Treasurer, with offices at 1621 Otis Building, Chicago.

Springfield Engineers Club—This club was organized at Springfield, Ill., Jan. 25, with a membership of 65 engineers and architects. President, Clifford Older, Bridge Engineer, Illinois Highway Commission; Vice-Presidents, P. C. McArdle, Principal Assistant Engineer, Illinois Highway Commission, and Frank H. Hamilton, City Commissioner; Secretary, J. R. Hughes.

Kansas Engineering Society—The seventh annual meeting was held at Topeka, Kan., Jan. 19 to 20. Papers were read covering a variety of subjects including steel sheetpiling, painting and protection of structural steel, office filing systems, Topeka sewer work, Kansas highways, city planning, irrigation, sewage purification, concrete construction, railway operation, municipal government and the regulation of public utilities.

Connecticut Society of Civil Engineers—The 31st annual meeting will be held in the Mason Laboratory, Sheffield Scientific School, Yale University, Feb. 9 and 10. The program includes many interesting papers on subjects such as the Alaska railway situation, port development at New London, the Yale Bowl, valuation of public utilities and others. The annual dinner will be held at the Hotel Taft on the evening of Feb. 9. The secretary is J. Frederick Jackson, New Haven, Conn.

American Concrete Pipe Association—At the annual meeting, to be held at the Auditorium Hotel, Chicago, Feb. 15 and 16 the following papers are to be presented: "Reinforced-Concrete Drain Tile," W. J. Schlick; "Cement Tile and the Drainage Engineer," J. H. Schmeltzer, Manteno, Ill.; "Concrete Sewer Pipe," L. R. Ferguson, Association of American Portland Cement Manufacturers; "Concrete Pipe for Railway Culverts," C. H. Cartlidge, Bridge Engineer, C. B. & Q. R.R., Chicago.

Idaho Society of Engineers—The sixth annual convention of the society was held at Boise, Idaho, Jan. 26 to 28. Papers were presented as follows: "Outfits, Transportation, Camps, etc., for Surveying Parties," by James Spofford; "A Railroad to Connect North and South Idaho," by Lee Highley; "Mining," by F. W. Callaway; "The Camera in Connection with Engineering Reports," by W. R. Heyde; "Cost of Power and Pumping for Irrigation, Minidoka Project," by Barry Dibble; "Requirements of Underwriters for Power Plant," by H. A. Seabright; "Problems and Construction of an Irrigation System," by W. O. Cotton; "Promoting an Irrigation Scheme," by W. H. Gibson; "Construction of an Irrigation System," by D. G. Martin, Edward Hedden, F. A. Wilkie, S. M. French and W. G. Sloan.

Albany Society of Civil Engineers—The annual banquet and election of officers was held Jan. 26. In the absence of the President, Frank R. Lanagan, who is on a trip to Porto Rico, Harvey O. Schermerhorn presided. The following officers were elected for 1915: President C. C. Covert, District Engineer, U. S. Geological Survey; Secretary, Ernest G. Raynor; Treasurer, Wilber E. Anderson. At the dinner Arnold G. Chapman, recently Deputy State Engineer, was the toastmaster. Speeches were made by R. E. Horton, Consulting Engineer, of Albany, who gave many interesting reminiscenses of his experience in hydraulic engineering; Maj. M. J. McDonough, Corps of Engineers, U. S. A., who paid a glowing tribute to the civil engineers' contribution to progress and civilization; Geo. A. Ricker, Deputy State Highway Commissioner; Frank M. Williams, State Engineer, and William R. Hill, Consulting Engineer, of Albany.

Appliances and Materials

A Non-Returnable Cloth Cement Sack

The Universal Portland Cement Co. announces that it is now offering dealer-customers a new type of cement package in the form of a non-returnable cloth sack. After a great deal of study and experiment it has succeeded in working out a sack that can be sold for 5c. each, and which at the same time should stand anywhere from five to six times as much rough handling as the average paper bag.

The company is now offering these sacks in an experimental way, with the idea of seeing whether or not they will be of assistance in solving the sack problem as far as the retail dealer in cement is concerned. The intention is to put the retail dealer in portland cement on the same basis as the retail dealer in flour, or any other commodity that must be sold in packages—that is to say, he will be furnished with a package that he can sell outright and with which he need not bother further. The initial price of these packages will be rather high as compared with paper bags, but they will represent no inconsiderable salvage value to the class of trade getting them. A farmer now buys four sacks of cement in paper and has nothing to show for the 10c. package charge that he paid. By paying 10c. a barrel more, he can get four sacks of cement in one-trip cloth sacks which will be available for a number of later uses.

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Road Scarifier Attachments

A scarifier attachment for road scrapers has recently been put on the market by J. D. Adams & Co., Indianapolis, Ind. The accompanying view shows one of the outfits. The device has been thoroughly tested under all conditions of road surface from clay to bituminous macadam, with entire success, it is claimed, provided the operator fully understands the machine and its adjustments. Those scrapers which are equipped with wheel-leaning adjustments permitting the wheels to oppose the side thrust on the inclined scarifier frame, give the best satisfaction in heavy work.

ADAMS ROAD GRADER EQUIPPED WITH SCARIFIER

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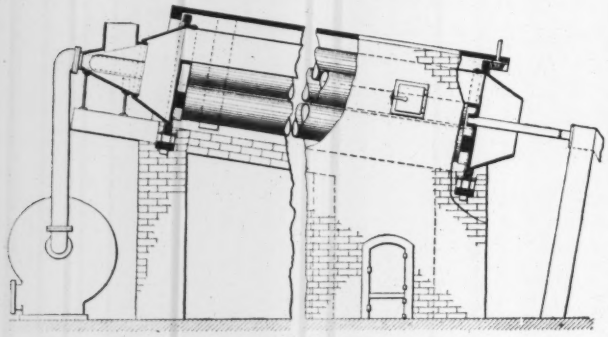
Multiple-Tube Rotary Dryers

A dryer for solids is shown in the accompanying illustration with five tubes fixed in steel supporting rings. This tube unit is supported, inclined at about 70°, on rollers, with a ball thrust bearing at the lower end, and is rotated through a bevel gear on the lower bearing ring. The tubes lead from a receiving chamber. The lower ends of the tubes open into a conical delivery chamber but restraining plates leave only that area of delivery opening found best for the particular material in hand. As the tube unit rotates, each tube takes in and discharges a desired amount of fresh material. A fan draws air up through the tubes and over the contents to assist in the drying. The tube unit is placed in a bricked flue over a gas or coal furnace so that hot gases are made to circulate around each tube before discharge. Steel co... plates facilitate access and removal. Heat absorption is fa... cilitated by the number of tubes and gases leave at 200° F. T... center of gravity of the load of material is kept close to the center of the dryer and the fall is only 15 in. so that wo... done, power required and wear incurred are decreased ov... single-tube dryer designs. The large heating surface and air movement through the tubes are claimed to result in operation at about one-third usual costs. A similar design has been made for liquid and semiliquid materials like sludges, tankage, garbage, molasses, etc. The drying chamber, however, is sealed at both ends. At the feed end a central pipe carries vapors and gases to a deodorizing tank or to the furnace; this pipe is used to feed in liquids. Semiliquids are introduced after opening the end of the feed chamber. At the discharge end a quick-opening double door closes all the tubes. The designs shown are known as the Houghton and are made by the Koller Boiler and Tank Co., Philadelphia.



A MULTIPLE-TUBE ROTARY DRYER

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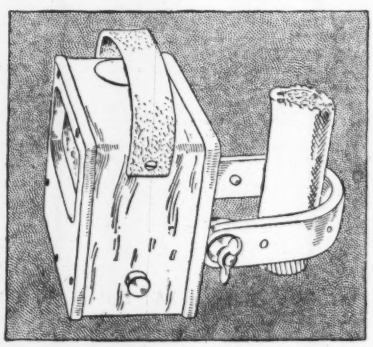
Direct-Current Cable-Testing Ammeter

An ammeter for reading the flow in a direct-current cable without actually connecting the instrument into the circuit was recently placed on the market by the Metropolitan Engineering Co., 30 E. 42d St., New York City. The instrument has the familiar D'Arsonval arrangement of a coil moving (restrained) in a magnetic field. Ordinarily the field is constant and the coil deflection dependent on a variable current through it. For this ammeter the situation is reversed; the current is held constant and the field allowed to vary.



A soft iron U-bar is placed around the cable whose current is to be measured and secured by wing nuts to two studs on the back of the instrument case. These studs are parts of the magnetic pole pieces between which is the moving system. The magnetic condition of the loop depends on the current in the cable and the coil deflection in turn on the magnetic field (after current is sent through the coil from a small dry-cell connected through a high resistance).

The utility of the instrument in measuring current without interrupting service on a cable is obvious. It has various other fields of usefulness like measuring stray direct current in dead cable sheaths or in alternating-current conductors. The standard size is 400 amp. capacity; special sizes run up to 1000 amp. The inventor is F. W. Morris, Electrical Engineering Department, Edison Electric Illuminating Co., Brooklyn, N. Y.